# DEFECTS AND FAILURES OF SOVIET STRUCTURES

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# DEFECTS AND FAILURES OF SOVIET STRUCTURES

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# INTRODUCTION

The Soviet engineers have, of necessity, designed and erected many buildings in the course of their post World War II activities. Some of these buildings have proved to be of faulty construction.

In 1956, the Soviet magazine "Stroitel'" (The Builder) published several photographs of unsoundly built structures as well as a series of cartoons making fun of the shortcomings of the Soviet building industry. But the Soviet government and "the party" did not choose to see the matter in too humorous a light.

Resolutions calling for improvement of structural designs, better organization and execution of construction work and higher quality of building materials were adopted by the government in 1955. The XX Congress of the Communist Party of the Soviet Union, convened in Moscow in February 1956, demanded emphatically that the building industry raise its standards.

Apparently reflecting the government's concern, a brochure and a few articles appeared in trade literature on the subject of structural failures in the Soviet Union (See bibliography).

Based on the above material, a description of some specific cases of these failures and their causes is made the subject of the report that follows.

The material indicates that defects and failures of Soviet structures have not been confined to any specific locality nor to any specific type of structure; they have occurred in such widely separated regions as White Russia, Moscow, Krasnoyarsk (Siberia), and appeared in both residential and industrial structures.

The report follows structural rather than geographic lines. The subject matter is therefore divided in two parts:

PART I. FAILURE OF RESIDENTIAL STRUCTURES

PART II. FAILURE OF INDUSTRIAL STRUCTURES

With two exceptions, each chapter within this subdivision covers a specific case of failure.

In preparation of individual chapters, an attempt was made to:

1) describe the structures under consideration from their foundations up whenever possible;

2) note in every case the construction time-table in view of the fact that construction methods and the behavior of materials are affected by the seasons of the year, particularly in the Soviet Union;

3) record in every case all available structural data, whether they have direct bearing upon a particular case of failure or not (such data may well be used in the analysis of similar structures when, as frequently happens, no reliable data on those under study are available);

4) ascertain whether the failure in question was primarily due to inadequancy of design, unsatisfactory materials, or carelessness on the part of the builders;

5) describe whenever possible the methods, suggested or actually used, in repair or reconstruction of the damaged structures.

Drawings and photographs pertaining to each failure are placed at the end of the chapter which describes such failure.

# PART ONE

# RESIDENTIAL STRUCTURES

CHAPTER I

# PARTIAL COLLAPSE OF A RESIDENTIAL STRUCTURE (MOSCOW)

## Location

44-46 Krasnoarmeyskaya Street, Moscow.

## Structure

Residential five-story brick structure with load-bearing exterior walls and interior piers located as shown in plan on Plate 1, fig. a.

## Construction

The structure was planned with a cellar conforming to one of the standard designs prepared by the SAKB APU Mosgorispolkom.* Actually, the building was erected without the cellar.

Foundations. Foundations are of precast reinforced concrete blocks.

Walls. The outside walls are built of seven-cell hollow ceramic blocks. Wall thickness is 51 cm. (20.1 in.); this figure includes a one-stretcher thickness of the facing made of silicate brick.

Interior piers. Interior piers are of Mark 100 brick on Mark 50 mortar; they are reinforced with steel mesh inserted in each bed joint, and their cross section varies from floor to floor as indicated in the following table:

| Floor No. | Pier cross section | | Pier mesh reinforcement | | | |
|---|---|---|---|---|---|---|
| | | | thickness | | mesh | |
| | cm. | in. | mm. | in. | cm. | in. |
| 1 | 77 x 77 | 30.3 x 30.3 | 5 | 0.197 | 5 x 5 | 1.97 x 1.97 |
| 2 | 64 x 64 | 25.2 x 25.2 | 4 | 0.158 | 6 x 6 | 2.36 x 2.36 |
| 3 | 64 x 51 | 25.2 x 20.1 | 4 | 0.158 | 6 x 6 | 2.36 x 2.36 |
| 4 | 51 x 51 | 20.1 x 20.1 | 4 | 0.158 | 6 x 6 | 2.36 x 2.36 |
| 5 | 51 x 51 | 20.1 x 20.1 | — | — | — | — |

*Building and Architechtural Construction Bureau of the Architectural Planning Administration, City of Moscow Executive Committee.

Beams. The rectangular cross beams are of precast reinforced concrete; 16 x 60 cm. (6.30 x 23.6 in.) in section; one end rests on piers, the other on outside walls.

Floors. The floors are of precast reinforced concrete slabs with hollow cylindrical cells. The slabs rest on cross beams, except for the end parts of the structure, where one end of the slab rests on cross beams, the other on the wall.

Pier-cross beam-floor slab joint. This joint is shown on Plate 1, fig. c. It should be noted that:

1) floor slabs fully cover each lift of the pier;

2) the beams rest on 38 x 25 cm. (15.0 x 9.84 in.) reinforced concrete bearing pads 14 cm. (5.51 in.) thick.

Construction time table. The most important part of the construction work was done during the fall-winter season of 1955-1956. Foundations were laid in the fall of 1955. Walls and piers of the 1st and 2nd floors were erected in January-February; those of the 2nd and 3rd floors in March, 1956. The 5th floor was completed during April-May of the same year.

Brickwork by freezing method. The walls and piers of the 1st, 2nd, 3rd, and 4th floors were erected in temperatures considerably lower than 32°F by the so-called "freezing method". This method is defined by a competent Soviet authority* in the following terms:

> "Freezing of masonry work is effected in winter time. The mix must have a temperature of from +8° to +12° C (46.4 - 53.6°F) when applied, water and ingredients being heated for this purpose. The fresh masonry work freezes and remains frozen during the entire cold period. In the spring, when thaw comes, the setting process (in the masonry work) continues. The strength of the masonry work laid by freezing method amounts usually to about 70% of the strength of the masonry work erected under normal conditions".

## Failure of Two Interior Piers and Its Consequences

On 17 May 1956, the pier at the intersection of axes 8 and B (Plate 1, fig. a) apparently collapsed first, then the pier at the intersection of axes 7 and B. This was followed by the collapse of all pier and floor construction above them. In the middle bay between axes 7 and 8 everything went clear down to the ground; in two adjacent bays, the broken panels remained mostly suspended in the air, hanging by their reinforcement embedded in the walls. Because of the fall of the attic floor, the cornice brickwork was somewhat lifted, became warped and developed vertical and horizontal cracks. The beams were torn out of their wall joints, leaving holes in the walls. The outside walls, however, remained standing.

Photograph of the failure is shown on Plate 2.

*L. K. Martens, Tekhnicheskiy Slovar', 1939, p. 538, T9.M25

## Causes of Failure

Post-failure examination of the structure, laboratory tests and recalculations disclosed the following facts as regards both materials and construction work.

1. Materials

a) Brick, quality adequate.

b) Mortar, quality below specifications (Specifications called for Mark 50 for summer conditions; Mark 25 for winter conditions after thawing and setting of mortar). Its strength was found to be 10-20 kg/cm$^2$ (142-284 lb/in$^2$).

c) Reinforcement of the interior piers did not conform to the specifications. The specifications called for 5 x 5 cm. (1.97 x 1.97 in.) steel mesh 5mm (0.197 in.) thick; instead, the builders used 10 x 10 cm. (3.94 x 3.94 in.) and 12 x 12 cm. (4.72 x 4.72 in.) mesh 4 mm (0.158 in.) thick. Moreover, it was not placed in every bed joint as required. As a result, the amount of reinforcement in the piers was reduced from 1% to 0.33%. The weakening effect of the above factors upon the brickwork may be surmised from the following table:

| Kind of Brickwork | Ultimate Compressive Strength | | | |
|---|---|---|---|---|
| | According to design | | Actual | |
| | kg/cm$^2$ | lb/in$^2$ | kg/cm$^2$ | lb/in$^2$ |
| Reinforced, laid under summer conditions. | 63.0 | 895 | 32.5 | 462 |
| Reinforced, laid under winter conditions after thawing and setting of mortar. | 46.0 | 654 | 29.5 | 419 |

2. Construction Work.

a) The gaps between the floor slabs and pier brickwork at the beams were not always properly filled. As a result, the floor slab load passed to the beams and beam bearing pads on the piers. With an uneven load distribution over the cross section of the pier (a part of the cross section being totally inactive because of the unfilled gaps), the brickwork under the beam bearing pads became overstressed; the compressive strength at that point amounted to 36 kg/cm$^2$ (511 lb/in$^2$) while the actual ultimate strength of the brickwork was only 29.5 kg/cm$^2$ (419 lb/in$^2$).

b) The sides of the floor slabs cutting into the pier and resting in part on the beam were crushed. Recalculations indicated that in the transfer of the slab load through the beam alone, the concrete between the hollow cells of the slab was under a stress of 243 $kg/cm^2$ (3,450 $lb/in^2$). The sides of the slabs resting on the beams of the first floor were taking up the weight of construction of the floors above them.

c) In some beam-pier joints a horizontal shift of beams was allowed to the extent of 12 cm (4.72 in.); thus, additional moments were developed. Photograph of such a beam is shown on Plate 3.

d) Assembly of foundation blocks was found to be faulty. The foundation cushion at the footing was made up of three 80 cm. (31.5 in.) instead of two 120 cm. (47.2 in.) blocks as required. The blocks were not covered with a leveling layer. The blocks of the second row, perpendicular to the first, did not rest soundly on the lower blocks. Whatever the tilting angle, additional moments could develop.

Considering the above facts it may be said the failure of the structure was due to:

1. failure of the pier brickwork under the beam bearing pad on the first floor;

2. failure of slab sides resting on beams.

Reconstruction

The two ruined piers were replaced with two monolithic reinforced concrete columns. Channel steel was used as reinforcement (this is explained by the necessity of meeting an early reconstruction deadline); concrete was poured as the reinforcement assembly was finished on each floor.

The foundations above the footings were strengthened by reinforced concrete casings on the assumption that this would ensure a more uniform transfer of forces to the footings. Column anchor bolts were embedded in the upper part of the casing.

Vertical section of a strengthened foundation is shown on Plate 4.

Remaining brick piers were strengthened at the four corners with 75 x 75 x 8 mm (2.95 x 2.95 x 0.315 in.) steel angles held together with 60 x 8 mm (2.36 x 0.315 in.) welded strips spaced at 50 cm. (19.7 in.) along the pier. The piers were then covered with steel mesh and plastered. The clearance between the floor slabs and the pier brickwork was to be carefully filled with tough cement and shimmed with pieces of steel plate.

Plan and vertical section of a reinforced brick pier are shown on Plate 5.

The fallen precast reinforced concrete beams and slabs were replaced with steel beams and presumably solid precast reinforced concrete slabs respectively. The other parts were reconstructed according to the original design.

## Note

In considering the causes of failure the following may be noted:

1) Weakness of the mortar was not due to its low quality but rather to the fact that the wrong kind of mortar was used.

2) Granted that the pier-cross beam-floor slab joints were badly constructed by the builders, still the fact remains that the design of the joint is of doubtful soundness.

3) In 1956, TsNIPS (Central Scientific Research Institute for Industrial Construction) ran some tests on hollow slabs embedded in brickwork, the slabs being of the same kind as those used in the structure under consideration. It was found that:

a) With brickwork mortar strength of 70 $kg/cm^2$ (995 $lb/in^2$), the slabs failed at the load of 260 $kg/cm^2$ (3,700 $lb/in^2$);

b) With the strength of mortar 15 $kg/cm^2$ (213 $lb/in^2$), slabs failed at 193 $kg/cm^2$ (2,740 $lb/in^2$).

Apparently, this information was unavailable to or neglected by the designers of the damaged structure. At any rate, with the actual mortar strength of 10-20 $kg/cm^2$ (142-284 $lbs/in^2$) in the piers and 243 $kg/cm^2$ (3,450 $lb/in^2$) stresses in the slabs that failed, the slab ends resting on beams of the first floor could be crushed by the load of construction above them.

Thus, behind the failure, there is an inadequate design and negligence on the part of the builders.

## Source

Moscow TsINIS. Causes of Structural Failures, pp. 17-24.
TH 3401.M7

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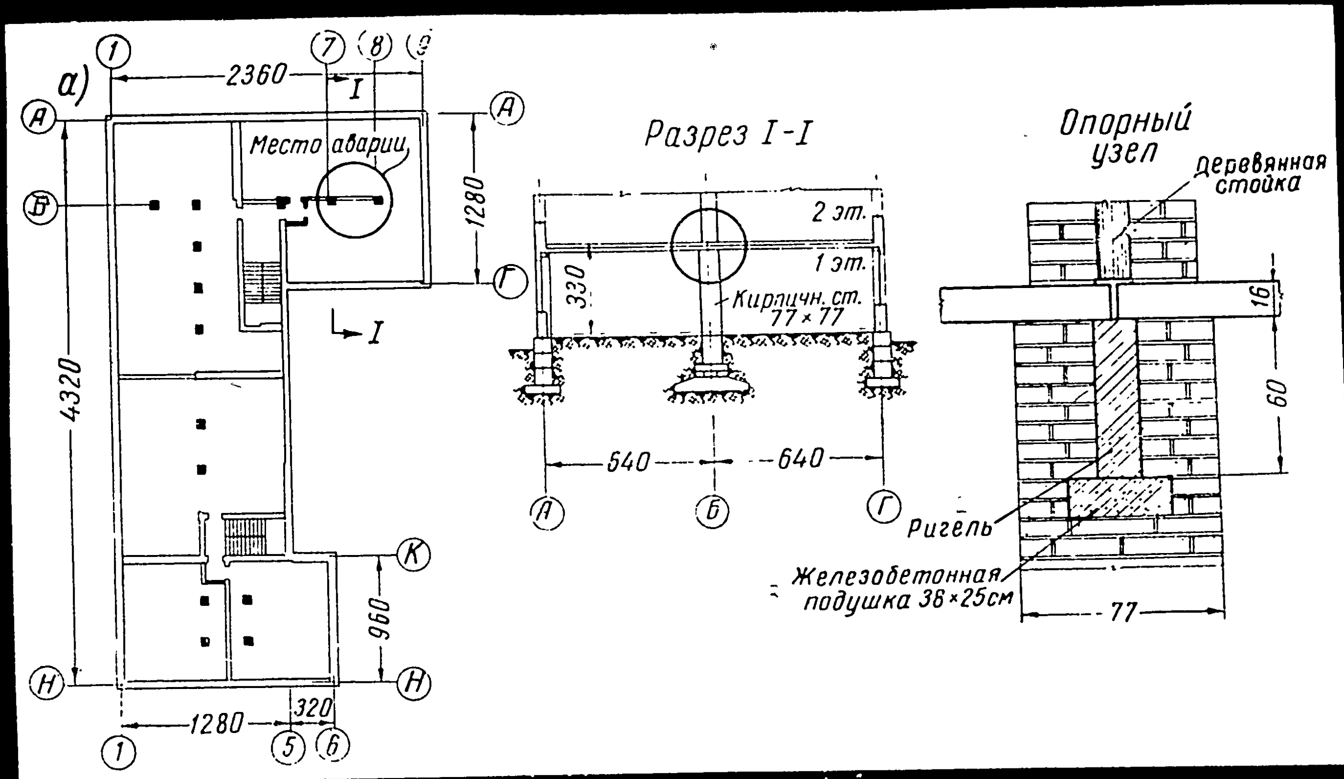


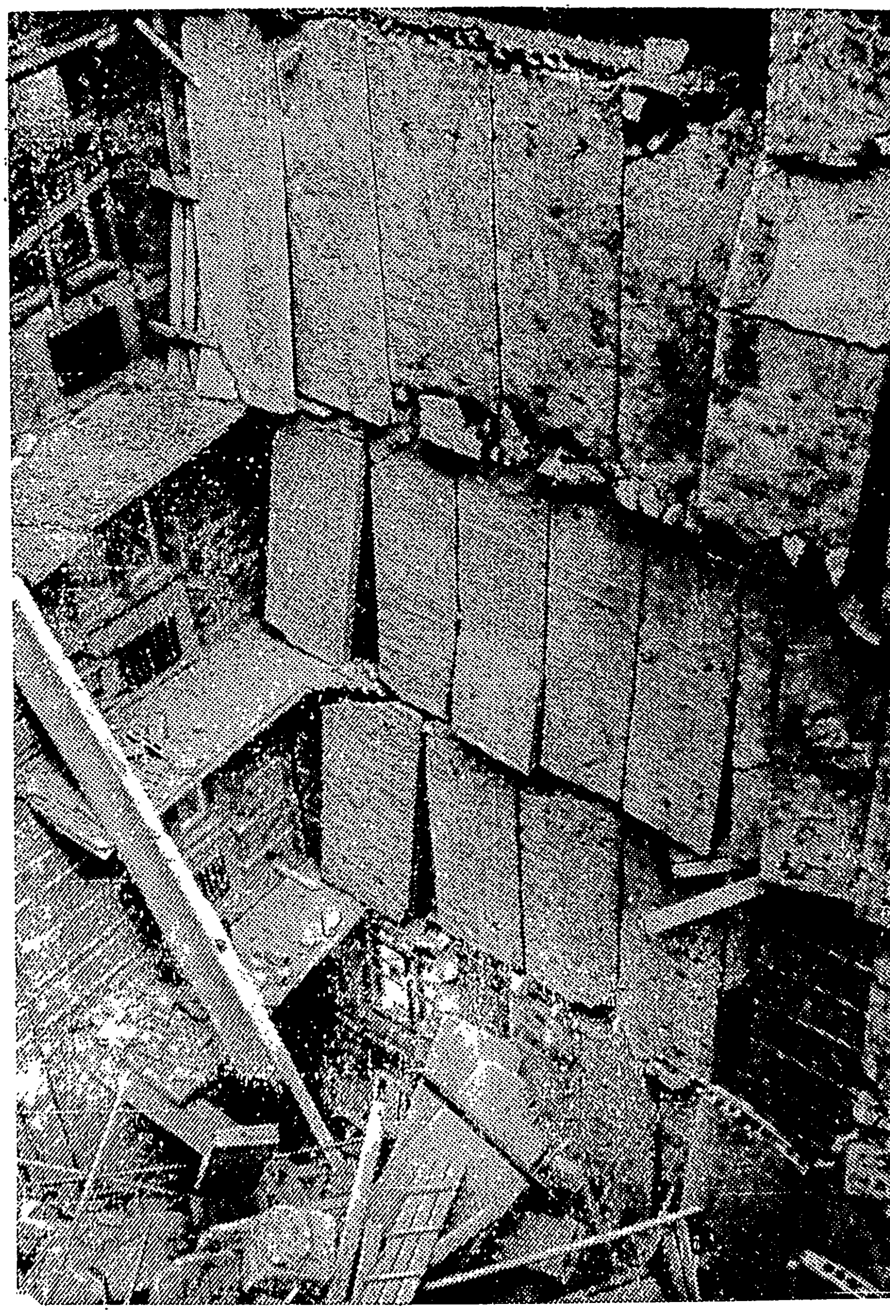

a. Plan. b. Vertical section, 1st floor.
c. Joint of pier with beam and floor slabs (1. Wooden post. 2. Beam. 3. RC bearing pad 15.0 x 9.84 in.)

The collapsed piers are marked with a circle.

RESIDENCE No. 44-46 KRASNOARMEYSKAYA STREET, MOSCOW.
Source: Moscow TsINIS, Causes of Structural Failures, p. 18 (TH 3401.M7)

Conversion Table

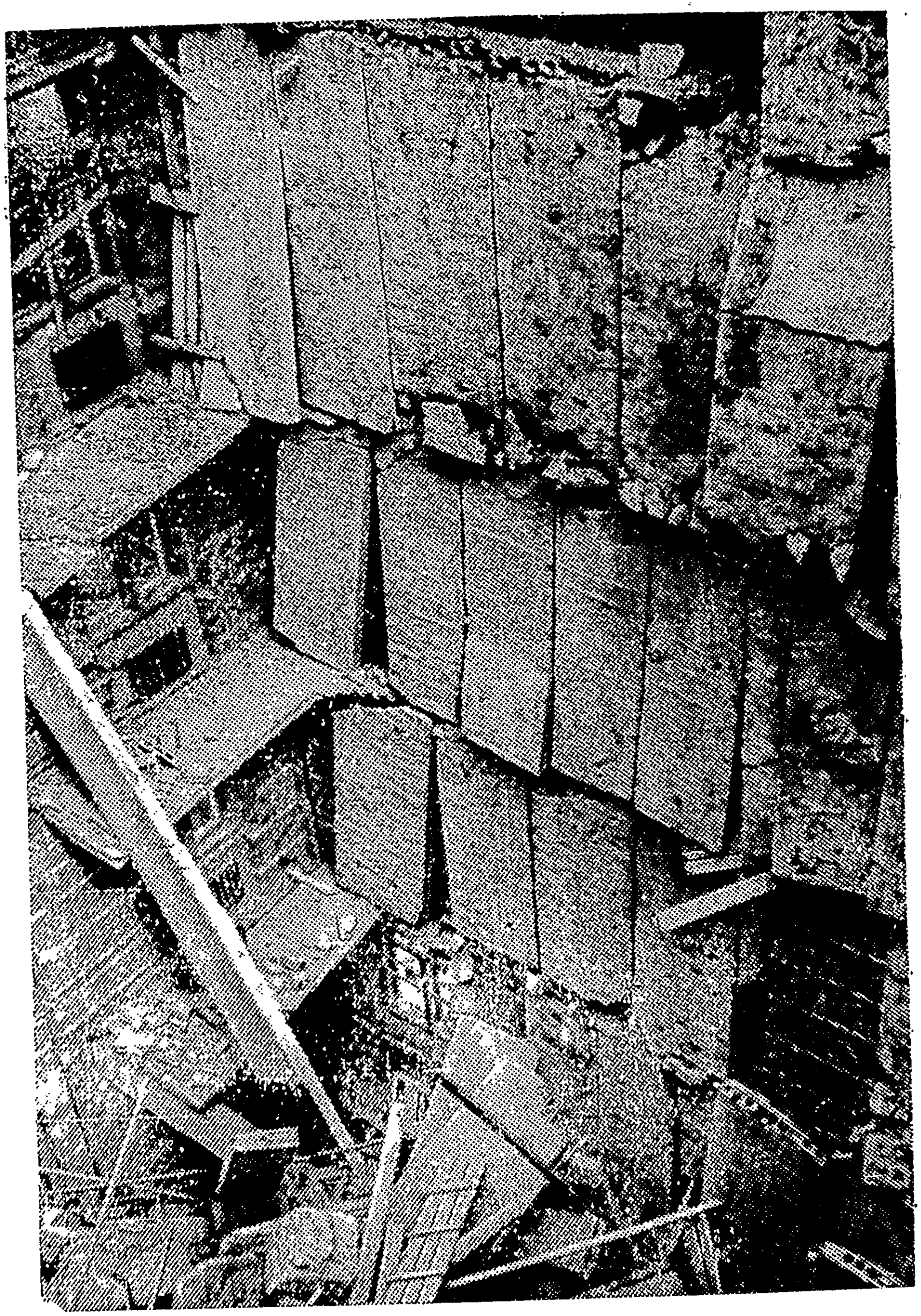
| cm. | ft. |
|---|---|
| 16 | 0.525 |
| 60 | 1.97 |
| 77 | 2.53 |
| 320 | 10.5 |
| 330 | 10.8 |
| 640 | 21.0 |
| 1,280 | 42.0 |
| 2,360 | 77.5 |
| 4,320 | 141.7 |



Partial view of the collapsed structure.

RESIDENCE NO. 44-46 KRASNOARMEYSKAYA STREET, MOSCOW.
Source: Moscow TsINIS. Causes of Structural Failures, p. 19. (TH 3401.M7)

PLATE 2



Partial view of the collapsed structure.

RESIDENCE NO. 44-46 KRASNOARMEYSKAYA STREET, MOSCOW.
Source: Moscow TsINIS. Causes of Structural Failures, p. 19. (TH 3401.M7)

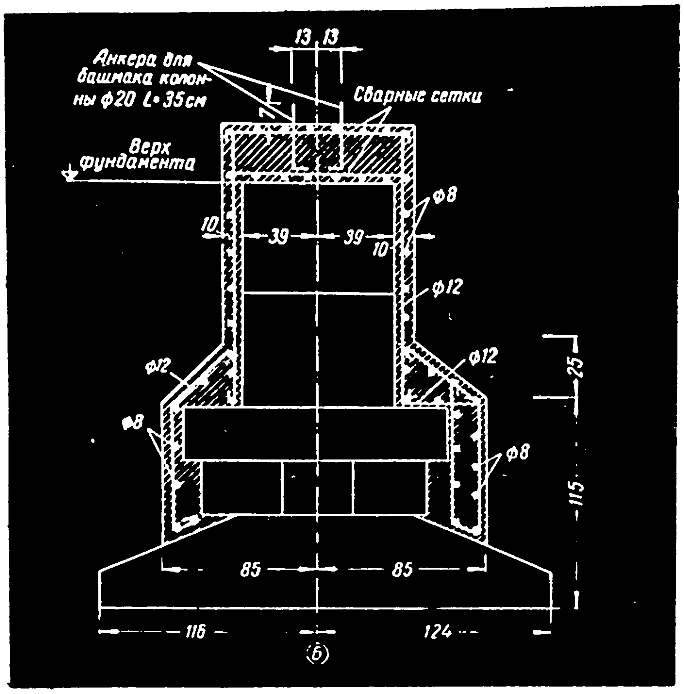
PLATE 2

Eccentric beam-pier joint

RESIDENCE No. 44-46 KRASNOARMEYSKAYA STREET, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 22. (TH 3401.M7)

PLATE 3

POOR ORIGINAL




1. Column shoe anchors, d=0.787 in; l=13.8 in.
2. Foundation Top
3. Welded mesh

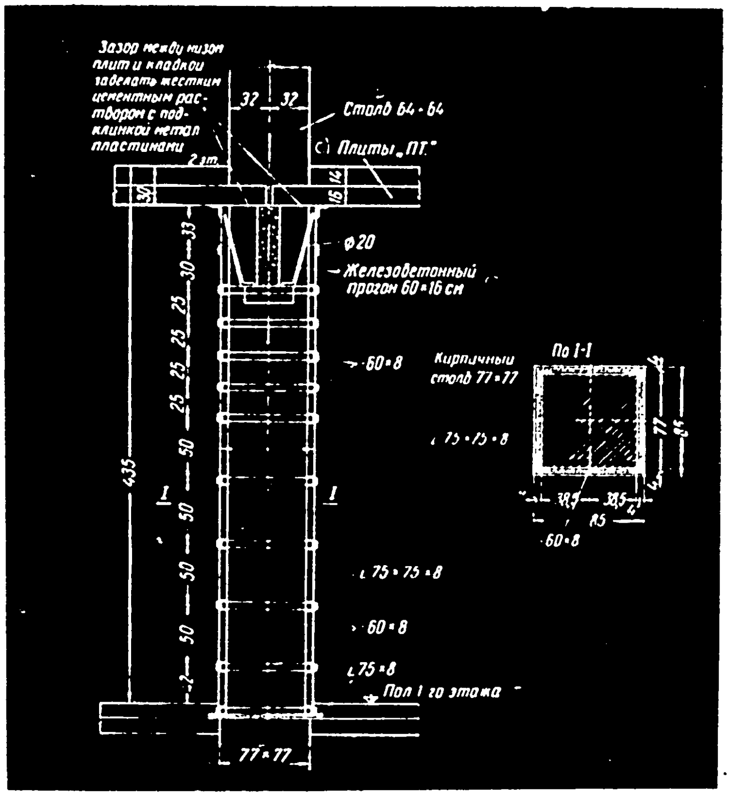
Strengthening of foundations with reinforced concrete casings.

Conversion Table

| mm. | in. | cm. | in. |
|---|---|---|---|
| 8 | 0.315 | 13 | 5.12 |
| 12 | 0.472 | 25 | 9.84 |
| | | 39 | 15.4 |
| cm. | in. | 85 | 33.5 |
| 7 | 2.76 | 115 | 45.3 |
| 10 | 3.94 | 116 | 45.7 |
| | | 124 | 48.8 |

RESIDENCE No. 44-46 KRASNOARMEYSKAYA STREET, MOSCOW.
Source: Moscow TsINIS. Causes of Structural Failures, p. 23. (TH 3401.M7)

PLATE 4




**Conversion Table**

| mm. | in. |
|---|---|
| 8 | 0.315 |
| 20 | 0.787 |
| 60 | 2.36 |
| 75 | 2.95 |

| cm. | in. |
|---|---|
| 4 | 1.58 |
| 14 | 5.51 |
| 16 | 6.30 |
| 25 | 9.84 |
| 30 | 11.8 |
| 33 | 13.0 |
| 38.5 | 15.2 |
| 42 | 16.5 |
| 50 | 19.7 |
| 64 | 25.2 |
| 77 | 30.3 |
| 85 | 33.5 |
| 435 | 171.2 |

a) Clearance between floor slabs and piers filled with tough cement and shimmed with pieces of steel plate
b) Pier (25.2 x 25.2 in.)
c) Floor slabs of "PT" type.
d) RC beam (6.30 x 23.6 in.)
e) Brick pier (30.3 x 30.3 in.)
f) First story floor

Strengthening of piers with steel angles and bands

RESIDENCE No. 44-46 KRASNOARMEYSKAYA STREET, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 24. (TH 3401.M7)

PLATE 5

# CHAPTER II

## COLLAPSE OF A PART OF A RESIDENTIAL STRUCTURE (MOSCOW)

### Location

16 Chaplygin Street, Moscow

### Structure

Residential five-story brick structure with outside longitudinal load-bearing walls and a single internal row of piers which support the cross beams.

Plan of the first floor is shown on Plate 6.

### Original Plan of Construction

The structure was to be erected in accordance with the standard design, Series P-01-13, worked out by the Building and Architectural Construction Bureau of the Architectural Planning Administration, City of Moscow Executive Committee (SAKB APU Mosgorispolkom). The original design, however, was modified by the agency that did the actual construction work.

### Construction as Actually Carried Out

Cross section and some structural details of the building both as originally designed and as modified appear in drawings shown on Plate 7, figs. 1 to 6.

The drawings suggest that two important changes were made:

1) a cellar with a row of interior piers was added;

2) the construction of the "pier-cross beam-floor slab" joint was changed.

Modification does not seem to have affected other structural details. They are the following.

<u>Foundations</u>. The structure is erected on continuous precast reinforced concrete foundations.

<u>Walls</u>. Wall material is Mark 100 brick laid in Mark 50 mortar. Thickness of the outside walls is 51 cm. (20.1 in.) throughout; the inside walls are 38 cm. (15.0 in.) thick.

Interior piers. The piers are of Mark 100 brick laid in Mark 50 mortar. Cross sectional dimensions of the piers vary from floor to floor as follows:

| Floor: | Cross section in cm. | Cross section in inches: |
|---|---|---|
| Cellar | 77 x 77 | 30.3 x 30.3 |
| 1st floor | 77 x 64 | 30.3 x 25.2 |
| 2nd " | 64 x 64 | 25.2 x 25.2 |
| 3rd " | 51 x 64 | 20.1 x 25.2 |
| 4th " | 51 x 64 | 20.1 x 25.2 |
| 5th " | 51 x 51 | 20.1 x 20.1 |

Beams. The rectangular cross beams are of precast reinforced concrete; their dimensions are:

| | | |
|---|---|---|
| thickness | - 16 cm. | (6.30 in.) |
| width | - 60 cm. | (23.6 in.) |
| length | - 638 cm. | (20.9 ft.) |

One end of the beams rests on longitudinal walls, the other on interior piers.

Floors. The floors are constructed of precast reinforced concrete slabs; their dimensions are:

| | | |
|---|---|---|
| thickness | - 16 cm. | (6.30 in.) |
| width | - 120 cm. | (47.2 in.) |
| length | - 358 cm. | (11.8 ft.) |

One end of the panels rests on the cross beams, the other on the transverse walls.

Pier-cross beam-floor slab joint. On every floor, the tops of interior piers provide support for both the floor slabs which fully cover them at this point and for the cross beams. The original design envisaged a reinforced concrete socket under every cross beam and at least a partial reinforcement of piers with steel mesh. (Plate 7, fig. 3). Later, it was decided that, in the projected cellar, the reinforced concrete sockets should be replaced with 38 x 38 cm (15.0 x 15.0 in.) reinforced concrete bearing pads. In the course of construction, however, these pads were replaced with 35 x 25 cm. (13.8 x 9.84 in.) steel plates 12 mm. (0.472 in.) thick. Moreover, the pillar brickwork was not reinforced with steel mesh. (Plate 7, fig. 6).

Construction time table. Earthwork and foundations, walls and piers of the cellar were completed in the fall of 1955 in temperatures above 32° F. The work on the rest of the building, including the 5th floor, was done during the winter of 1955-1956 in temperatures considerably lower than 32°F. The floors of the 2nd, 3rd, 4th, and 5th stories, completed under winter conditions, were strengthened by wooden supports which were placed under cross beams beside the interior piers.

## Collapse of the Structure

The structure collapsed on 15 April 1956. Apparently, this happened as follows. First, the interior pier in the cellar collapsed (intersection of axes 10 and B, Plate 6). This was followed by the collapse of all pillars and floors above it. Under the weight of the fallen floor panels, the cross beams were cut off at their supports at the walls. Some panels went down, some remained hanging, held by their reinforcement still embedded in the walls. With the floors collapsed, a considerable portion of the front wall went down and the end wall became warped. The wreckage is shown on Plate 8.

## The Causes of Collapse

Two factors seem to have contributed to the collapse of the pier in the cellar. They were the following:

1) Uneven quality of the brick and mortar. The brick was certified to be of Mark 100 but much of it proved to be of Mark 50. Analysis of the mortar taken out of the brick joints had shown that its maximum compressive strength was 29 kg/cm$^2$ (412 lb/in$^2$) instead of 50 kg/cm$^2$ (710 lb/in$^2$). The load bearing capacity of the pier brickwork suffered accordingly.

2) Inadequacy of the pier-cross beam-floor slab joint. The idea of the reinforced concrete sockets and bearing-pads having been abandoned, the joints of the cross beams with the pier, were, in the end, made on steel ~~plates without~~ the steel mesh reinforcement of the underlying brickwork. Therein lies the other reason for the collapse of the pillar - local overstress at the steel plates. The area of these plates being some 60% smaller than that of the rejected reinforced concrete bearing pads, a considerable increase in local stress was to be expected. Post-failure recalculations have indicated that the local compressive stress amounted to some 35-40 kg/in$^2$ (498-569 lb/in$^2$) at the plates. This raises the question of the structural soundness of the entire pier-cross beam-floor slab joint. It seems that:

1) a more uniform distribution of load to the pier could be achieved through a reinforced concrete socket rather than through the steel plates or even reinforced concrete pads.

2) the floor slabs fully covering the top of the pier on one floor make the proper centering of the pillar on the next floor rather difficult; eccentric loading of the improperly centered pier would create bending moments, thus weakening the load-bearing capacity of the brickwork.

## Reconstruction

The collapsed portion of the structure was reconstructed as indicated in part on Plate 9.

The interior piers on all floors were replaced with 38 cm. (15.0 in.) thick walls with pilasters which provided support for the cross beams.

The end wall was strengthened on the outside by three 38 cm. (15.0 in.) thick brick diaphragms which extended to the end wall of the neighboring building.

Construction of the floors was lightened. Wood panel subflooring was introduced, on wood beams laid on cross beams and brick walls. Precast reinforced concrete slabs were used only around plumbing installations, and for the ground floor. The support for these panels was provided by reinforced concrete beams laid in the face of the walls in addition to the cross beams.

The interior piers in the cellar and on the first floor of the undamaged part of the structure were to be strengthened with steel angles.

Source

Moscow TsINIS. Causes of Structural Failures, pp. 12-17, TH 3401.M7

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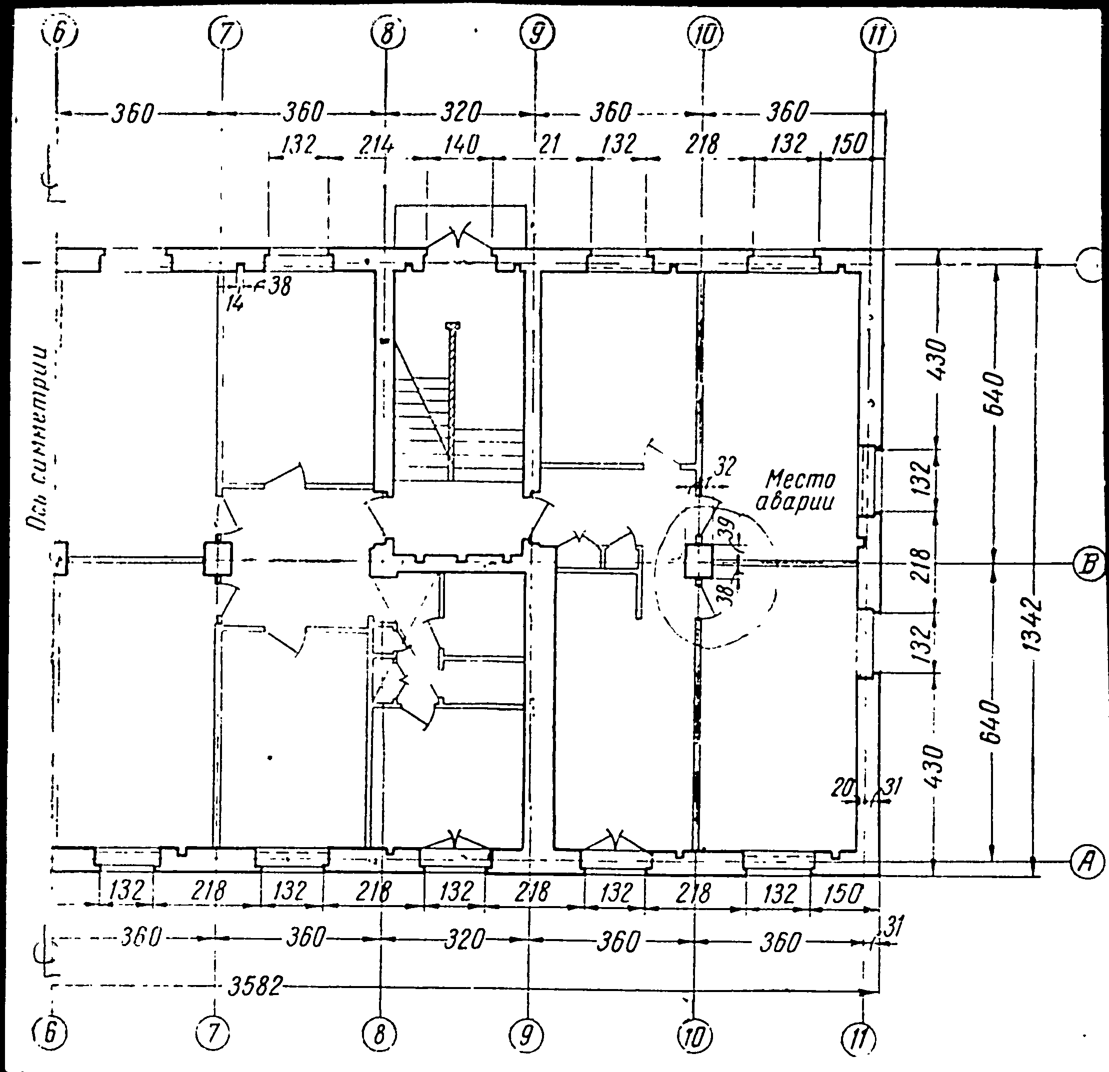



Plan of the 1st floor (Standard design, Series P-01-13)
The collapsed pier is marked with a circle

Conversion Table

| cm. | ft. | cm. | ft. |
|---|---|---|---|
| 20 | 0.656 | 218 | 7.15 |
| 31 | 1.02 | 320 | 10.5 |
| 38 | 1.25 | 360 | 11.8 |
| 132 | 4.33 | 430 | 14.1 |
| 140 | 4.60 | 640 | 21.0 |
| 150 | 4.93 | 1,342 | 44.0 |
| | | 3,582 | 117.6 |

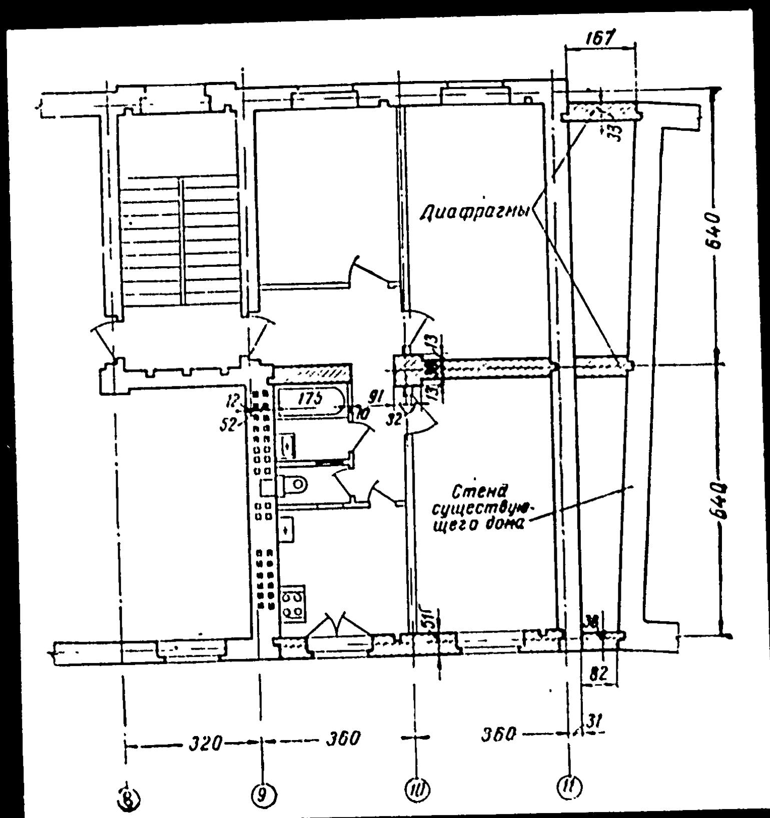
RESIDENCE No. 16 CHAPLYGIN STREET, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 13. (TH 3401.M7)

PLATE 6

General view of the collapsed part of the structure

RESIDENCE No. 16 CHAPLYGIN STREET, MOSCOW
Source: Moscow TsINIS: Causes of Structural Failures, p. 15 (TH 3401.M7)

PLATE 8




1. Brick shoring diaphragms
2. The wall of the neighboring building

Reconstruction: Plan of the walls.

Conversion Table

| cm. | in. | cm. | in. |
|---|---|---|---|
| 10 | 3.94 | 51 | 20.1 |
| 12 | 4.72 | 52 | 20.5 |
| 13 | 5.12 | 82 | 32.3 |
| 31 | 12.2 | 167 | 65.7 |
| 32 | 12.6 | 320 | 126.0 |
| 33 | 13.0 | 360 | 141.7 |
| 38 | 15.0 | 640 | 252.0 |

RESIDENCE No. 16 CHAPLYGIN STREET, MOSCOW

Source: Moscow TsINIS. Causes of Structural Failures, p. 16 (TH 3401.M7)

PLATE 9

# CHAPTER III

## FRONT WALL FAILURE IN A RESIDENTIAL STRUCTURE (MOSCOW)

### Location

53 Gertsen Street, Moscow.

### Structure

Residential four-story brick structure with outside load-bearing walls and interior longitudinal wall.

### Construction

Front elevation, plan and cross section of the front wall are shown on Plate 10.

Walls. Walls are of Mark 100 brick laid in Mark 50 mortar. Thickness of the exterior walls is 64 cm. (25.2 in.); the interior wall is 51 cm. (20.1 in.) thick. On the outside, the walls are faced with L-shaped ceramic tiles of the KG type with vertical sides 7 cm. (2.76 in.) thick.

Beams. Floors of the 2nd and 4th stories are supported by steel I-beams No. 40 (weight unspecified; see SES Report No. 1, Table 1.0253B) 6.8 m. (22.3 ft.) long; bay length is 3.6 m. (11.8 ft.). The beams rest at one end on the outside wall piers, with the other on the internal longitudinal wall. The floor of the 3rd story is supported by precast reinforced concrete beams of 60 x 16 cm. (23.6 x 6.30 in.) cross section.

Floors. The floors are of precast reinforced concrete panels of the PRT type. On the 2nd and 4th floors, the panels rest on the lower flanges of the steel I-beams; on the 3rd floor, presumably on the precast reinforced concrete beams.

Construction time table. Wall brickwork of the first three stories was erected by freezing method during the winter. By 18 April 1956, a part of the 4th floor wall was completed.

### Partial Collapse of the Front Wall

On 18 April 1956, the part of the wall around the main entrance collapsed, carrying down with it a three-bay length of the 1st, 2nd, and 3rd floor constructions.

### Causes of Failure

The follapse of the wall was due to the failure of two front wall piers flanking the front entrance.

The wall piers, of 126 x 64 cm. (49.6 x 25.2 in.) cross section, and 4.1m. (13.5 ft.) high, are shown in plan on Plate 10.

Apparently the piers failed through inadequate load-bearing capacity at the time of thaw, when not even temporary bracing was provided. The piers were further weakened by the following factors:

1) The bricklayers used chopped-off tops of hollow ceramic blocks to level their courses. These pieces, 20 mm. (0.787 in. thick were obtained on the site (See Plate 11, fig. 1).

2) The horizontal brick joints behind the ceramic facing were inadequately filled. Examination of such joints in the remaining piers disclosed that some of them were dry to a depth of 15-17 cm. (5.91-6.70 in.).

## Reconstruction

Main details of the reconstruction of the wall are shown in plan on Plate 11, fig. 2.

Cross section of the new wall piers was enlarged from 126 x 64 cm. (49.6 x 25.2 in.) to 130 x 64 cm. (51.1 x 25.2 in.). Part of their load was transferred to the adjoining 64 x 51 cm. (25.2 x 20.1 in.) pilasters which were erected under the beams.

## Note

Assuming that the piers were correctly calculated, it would seem that:

1) the designers failed to specify (and the builders managed to neglect) the measures to be taken (temporary bracing under the beams, for instance) at a critical moment when masonry structures enter the thawing period.

2) introduction of ceramic block tops (possibly cut unevenly and even cracked) in pier brickwork may have weakened them.

3) the faulty brick joints between the tile facing and the body of the piers weakened the piers considerably; the depth of dry joints was more than twice that allowed by the Soviet "Norms and Technical Requirements". Regardless of regulations, however, it is clear that such construction is subject to additional moments which reduce the load-bearing capacity of the piers.

One is inclined to conclude that in this case, builders' negligence rather than faulty design or materials was responsible for the failure of the wall.

## Source

Moscow TsINIS. Causes of Structural Failures, pp. 30-33. TH 3401.M7

Fig. 1.
Front elevation

Fig. 2.
Plan
Section 2-2

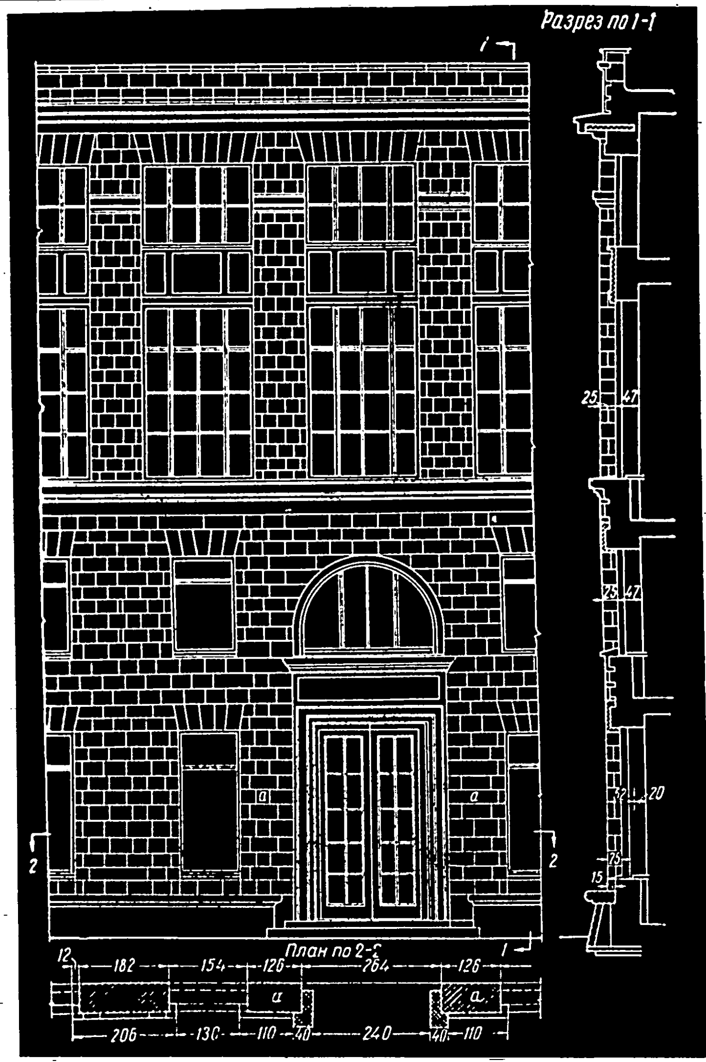



Fig. 3.
Vertical section;
Section 1-1

Plan, front elevation and section of the part of the wall that collapsed

Conversion Table

| cm. | in. | cm. | in. |
|---|---|---|---|
| 12 | 4.72 | 110 | 43.3 |
| 15 | 5.91 | 126 | 49.6 |
| 20 | 7.87 | 130 | 51.1 |
| 25 | 9.84 | 154 | 60.6 |
| 40 | 15.7 | 182 | 71.6 |
| 47 | 18.5 | 206 | 81.0 |
| 52 | 20.5 | 240 | 94.5 |
| | | 264 | 104.0 |

RESIDENCE No.53 GERTSEN STREET, MOSCOW

Source: Moscow TsINIS. Causes of Structural Failures, p. 31. (TH 3401.M7)

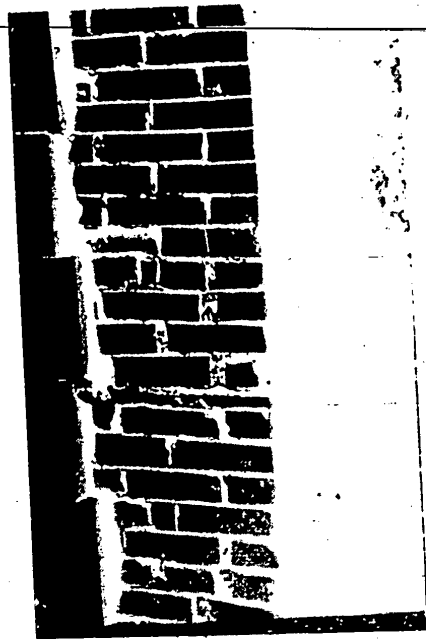


Fig. 1. Pier similar to those which collapsed. Top of a hollow ceramic brick may be seen in the brickwork.

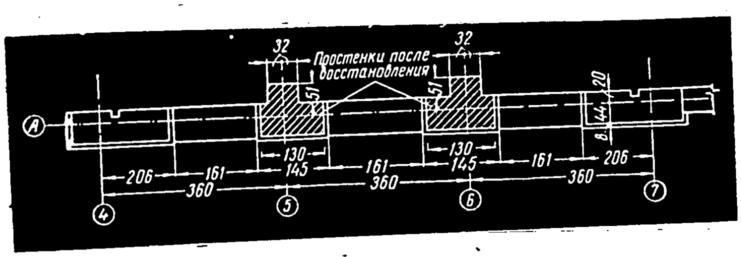



Fig. 2. Reconstructed wall piers. Plan of the 1st floor

Conversion Table

| cm. | in. | cm. | ft. |
|---|---|---|---|
| 8 | 3.15 | 130 | 4.27 |
| 20 | 7.87 | 145 | 4.76 |
| 32 | 12.6 | 161 | 5.28 |
| 44 | 17.3 | 206 | 6.76 |
| 51 | 20.1 | 360 | 11.8 |

RESIDENCE No. 53 GERTSEN STREET, MOSCOW

Source: Moscow TsINIS. Causes of Structural Failures, pp. 32-33 (TH 3401.M7)



PLATE 11

# CHAPTER IV

## COLLAPSE OF A PRECAST REINFORCED CONCRETE FRAMEWORK (UKRAINE)

### Location

Village by the Mikhailovskaya mine No. 12, Voroshilovgrad Oblast', Ukrainian SSR.

### Structure

Residential 36-apartment four-story precast reinforced concrete frame structure with a cellar. Some of its measurements are:

| | |
|---|---|
| Length | 49.12 m (161 ft.) |
| Width | 11.50 m (37.8 ft.) |
| Cubage | 9412.00 $m^3$ (332,000 $ft^3$) |
| Living area | 1095.00 $m^2$ (11,800 $ft^2$) |

Two settlement joints cut through its entire width and height, dividing it lengthwise in three 16 m (52.5 ft.) sections.

Front and side elevations of the structure are shown on Plate 12, fig. 1.

### Construction

Isometric view of the building is shown on Plate 12, fig. 2.

The structure is composed of 18 transverse four-story precast reinforced concrete frames which provide support for the precast reinforced concrete floor panels.

Construction of joints. Column and beam-column joints are made monolithic, by first welding the reinforcement, and then covering the weld with expanding cement.

Panel and beam-panel joints are reinforced with steel mesh and plates, and are also made monolithic.

Structural materials.

1) Besides the beams, columns and floor panels, the following structural members are also of precast reinforced concrete: stairways, stair-heads, cornices, outside wall panels and smoke and ventilation stack casings.

2) Precast reinforced concrete ribbed panels of the outside walls are insulated with foam concrete.

3) The walls between sections of the structure and between the apartments are of slag concrete blocks.

4) Partitions within the apartments are of gypsum panels.

5) Foundations and cellar walls are of rubble concrete.

## Total Collapse of the Framework

Some phases of the construction work may have been started some time in the latter part of 1953. By the end of July 1954, the following was accomplished:

1) foundations, cellar walls and ground floor were completed;

2) all 18 transverse frames were erected;

3) floor panels were partially in place;

4) on the 1st floor of the third section, a part of the between-section and of two between-apartment slag concrete walls were in the process of erection.

On 28 July 1954, the entire superstructure went down in the general direction of its longitudinal axis.

The debris is shown in photographs on Plate 13, figs. 1 and 2.

## Causes of collapse

The immediate cause of the accident was ascribed to the force of wind from the west which, as reported by the Voroshilovgrad airport meteorological station, reached the velocity of 14 - 18 m./s. (31 - 41 mph) on that day.

The real cause, however, should be sought somewhere else.

Erection schedule of the superstructure required that:

1) erection of the structural members of the first two floors be completed and all joints made monolithic;

2) only following this, the same procedure was to be applied to the next two floors; before the joints were made monolithic temporary bracing was to be provided.

This procedure was disregarded in practice.

Before completing the first two floors (that is, before the joints were made monolithic, the outside wall panels installed, and the internal walls built), the builders erected the members of the 3rd and 4th floors without making the joints permanent or even using temporary bracing. Prior to its collapse, the structure remained unfastened for 13 days! Strong wind was sufficient to bring it down.

## Note

The stiffness of the joints and the stability of the structure as a whole were apparently sufficiently ensured by the designers. Assuming that the quality of structural materials was adequate, the collapse of the structure was due to an extraordinary display of negligence on the part of the builders.

## Source

Moscow TsINIS. Causes of Structural Failures,
pp. 38-42. TH 3401.M7

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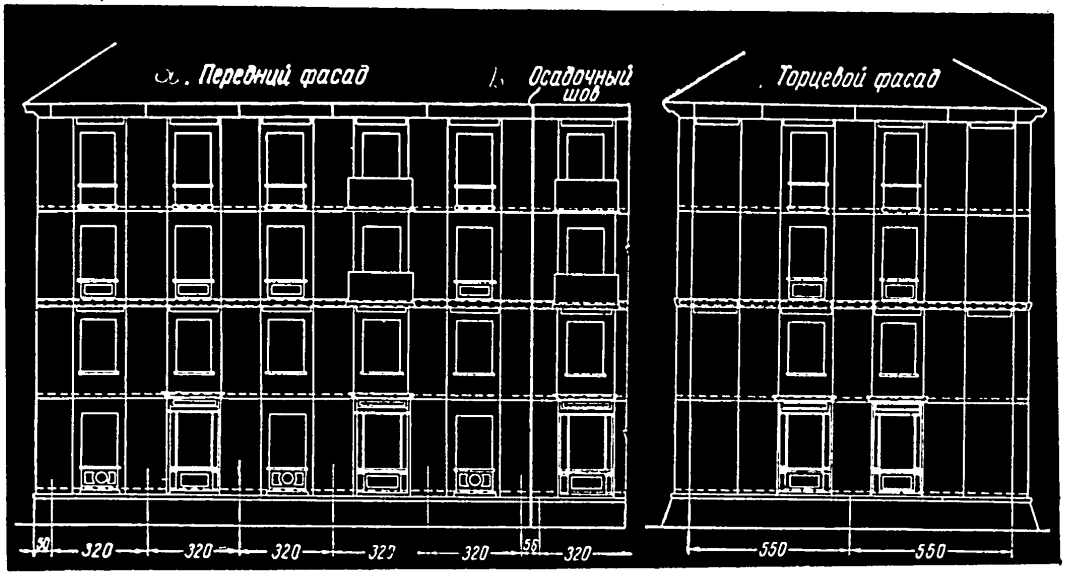


Fig. 1. Front and end elevations of the structure
a. Front elevation. b. Settlement joint. c. End elevation

Conversion Table

| cm. | in. | ft. |
|---|---|---|
| 50 | 19.7 | — |
| 56 | 22.0 | — |
| 320 | —— | 10.5 |
| 550 | —— | 18.0 |

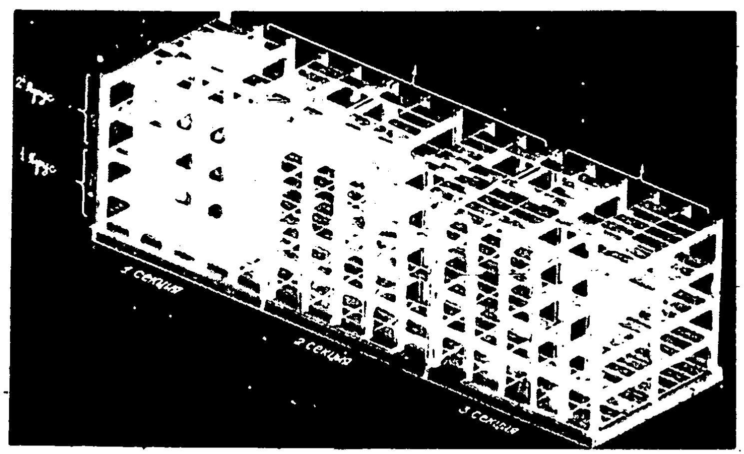


Fig. 2. Isometric view of the structure at the time of collapse

RESIDENTIAL PRECAST REINFORCED CONCRETE FRAME STRUCTURE AT A VILLAGE BY THE MIKHAYLOVSKAYA MINE No. 12, VOROSHILOVGRAD OBLAST', UKRAINIAN SSR

Source: Moscow TsINIS. Causes of Structural Failures, pp. 38-42 (TH 3401.M7)

PLATE 12

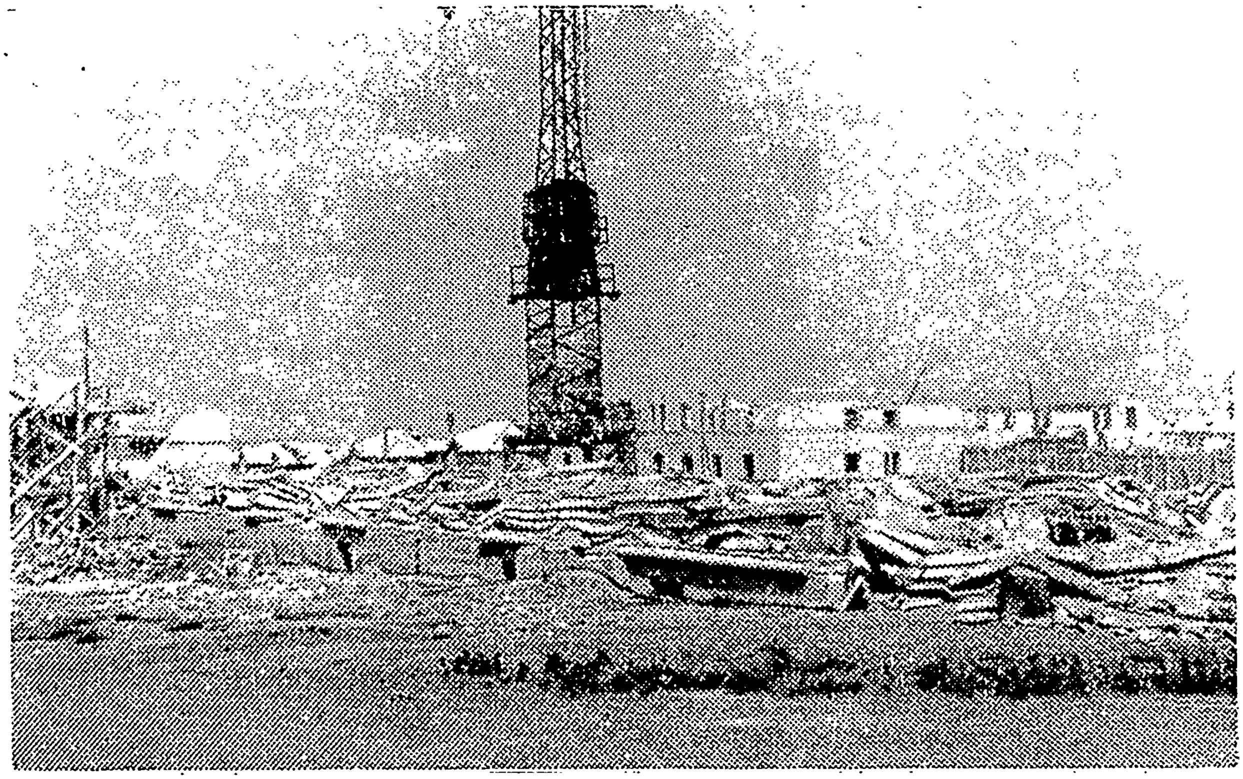

Fig. 1. View looking toward the front wall

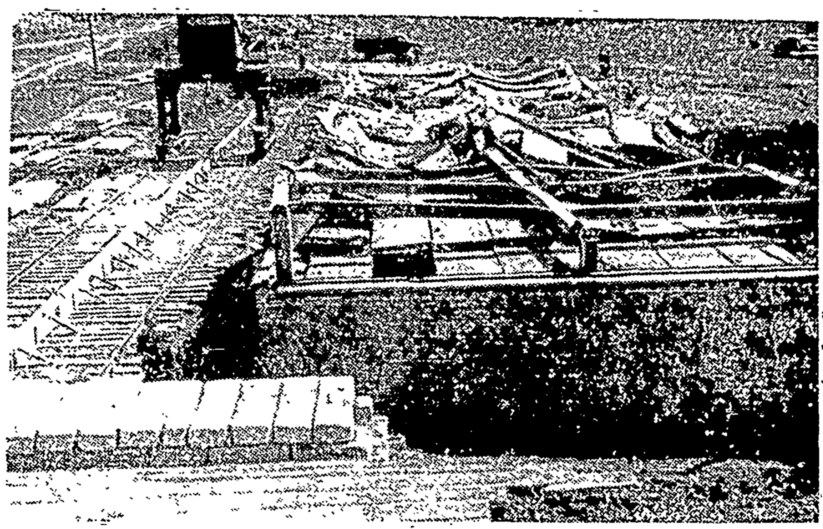

Fig. 2. View looking toward the end wall

GENERAL VIEWS OF THE DEBRIS AFTER THE COLLAPSE OF THE RESIDENTIAL STRUCTURES BY THE MIKHAYLOVSKAYA MINE No. 12
(Captions in the original seem to be interchanged)

Source: Moscow TsINIS. Causes of Structural Failures, pp. 38-42. (TH 3401.M7)

PLATE 133



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# CHAPTER V

## RACKING OF A LARGE-PANEL FRAMELESS RESIDENTIAL STRUCTURE (MURMANSK)

### Location

City of Murmansk

### Coordinates

Latitude - 68° 58' N; Longitude - 33° 05' E

### Structure

Four-story large-panel frameless residential structure with cellar.

### Construction

Plan and photograph of the structure in the process of erection are shown on Plate 14, figs. 1 and 2.

Overall dimensions:

| | |
|---|---|
| Width | 65.3 m (214 ft.) |
| Length | 13.8 m ( 45.3 ft.) |
| Height | 16.5 m ( 56 ft.) estimated |
| Height of a story | 3.3 m ( 10.8 ft.) |

Footings. Material: Mark 100 reinforced concrete.

| | |
|---|---|
| Thickness: | 0.20 m. (7.87 in.) |
| Width: under outside walls | 1.45 m. (4.75 ft.) |
| under partitions | 1.40 m. (4.60 ft.) |
| under stair partitions | 1.25 m. (4.10 ft.) |

Reinforcement: 10mm (0.394 in.), spaced 0.2 mm (7.87 in.) transversely and longitudinally.

Foundation walls (Cellar walls): Continous monolithic concrete.

| | | |
|---|---|---|
| Wall thickness: | outside walls | 0.75 m. (29.5 in.) |
| | inside walls | 0.3-0.5 m (11.8-19.7 in.) |

Upper walls: Large room-size concrete panels.

| | | |
|---|---|---|
| Panel thickness: | | 0.3 m. (11.8 in.) |
| Panel composition: | outside reinforced concrete layer | 40 mm. (1.60 in.) thick |
| | middle foam concrete layer | 210 mm. (8.26 in.) thick |
| | inside reinforced concrete layer | 50 mm. (1.97 in.) thick |

The inner and outer reinforced concrete layers of each panel are joined by reinforced concrete ribs around the perimeter and around the window opening.

Vertical joint of exterior wall panels. These joints are made monolithic by welding the reinforcement and filling the joint with mortar or concrete. They are braced by reinforced concrete pilasters.

Partition panels. The partition panels consist of a reinforced concrete frame filled with slag concrete. They are 0.14 m. (5.51 in.) thick.

Floor slabs. The flat solid slabs are of Mark 200 reinforced concrete. They are 90 mm. (3.54 in.) thick. Slabs are joined monolithically by welding the reinforcement and filling the joint with mortar or concrete.

Reinforcement working stresses. These stresses are as follows:

| | |
|---|---|
| for round steel | 1,700 $kg/cm^2$ (24,200 $lb/in^2$) |
| for deformed bars | 2,400 $kg/cm^2$ (34,100 $lb/in^2$) |

Stairways and cornice. Both stairways and cornice are of precast reinforced concrete.

Roof. The roof has wooden rafters and purlins.

Roofing. The roof is covered with black steel sheets.

Geological conditions at the site. The structure was erected on soil composed of loam and sandy loam layers. Soil pressure was assumed to be 1.25 $kg/cm^2$ (2,560 $lb/ft^2$). Soil characteristics are the following:

| Soil Characteristics | Loam Layer (plastic) | Sandy loam layer |
|---|---|---|
| Natural moisture content | 20 - 32% | 19 - 27% |
| Volume weight (natural moisture) | 2.03 $t/m^3$ (126 $lb/ft^3$) | 1.23-1.53 $t/m^3$ (76.6-95.3 $lb/ft^3$) |
| Porosity | 0.781 (coefficient) | 39-49% |
| Plasticity | 9 - 13 | 4 |
| State of saturation | nearly complete | complete |

Test drillings indicated that ground water level in that locality was at the depth of 1-3 m. (from 3-10 ft.).

## Signs of racking

The erection of the building was apparently begun early in 1955 and completed early in 1956.

On 25 May 1956, first cracks were noted in the walls of the cellar. In the external wall (between axes 1-4, Plate 14, fig. 1), they were vertical and 5mm. (0.2 in.) wide; in the interior wall (between axes 1-4 and C) they were horizontal and also 5mm (0.2 in.) wide. Numerous cracks, up to 2mm (0.08 in.) in width, were discovered in the panel joints but none in the body of the panels.

Gypsum "screeds" applied to the cracks in the cellar walls and in the panel joints on the 4th floor registered cracks up to 1 mm. (0.04 in.) in width during a 10 day period of observation. The vertical cracks tapered off in the direction from the cornice toward foundations.

The "screeds" applied between 16 June and 18 June 1956, to the wall panel joints on the 2nd, 3rd and 4th floors registered cracks 0.1 mm (0.004 in.) wide in a 2 day period.

On 6 July 1956, extensive damage was observed (along axes 3 and 4, Plate 14, fig. 1) in vetical joints of the external and internal walls as well as in the transverse joints of the floor slabs. The joints of the cornice panels widened to 30 mm. (1.2 in.) beyond the clearance allowance (Plate 15, fig. 1). Inspection disclosed that reinforcement was torn out of concrete, and that the 1.2 in. gap in the cornice corresponded to a 1.2 in. crack in the attic floor slabs. Similar damage was observed in the panel joints of both longitudinal wall and partition. Attic floor slabs (along 3 and 4) moved apart a distance of 20 - 25 mm. (0.79-0.98 in.) This may be explained by the fact that reinforcement projections, i.e., the rods joining the adjacent floor slabs together, did not coincide; being placed under an angle they could not prevent the separation of the panels, (Plate 15, fig. 2).

Settlement of foundations. Examination disclosed that the soil under the foundations was sodden loam.

The settlement of foundations proceeded at the following rate:

| | |
|---|---|
| August 1956 | 12 mm./month (0.47 in./month) |
| October 1956 | 6.8 mm/month (0.27 in./month) |

The average settlement of foundations for the period of construction (assuming that it started in October 1955 when the erection of the first floor panels was begun) amounted to 120 mm (4.72 in.). Relative difference in the settling of foundations is approximately 0.001.

## Causes of Racking

The main causes underlying the racking of the structure were the following:

1. Compressibility of the soil under foundations was uneven.

2. Repeated wetting and freezing, and subsequent softening of the soil around foundations (axes 1-4) in the course of construction.

3. Freezing of the base of external walls and partitions.

4. Construction of the part of the structure where axes 1-4 are located lagged 2 stories behind the erection of the other parts of the building. The uneven loading of foundations may lead to uneven settlement of foundations and result in additional stresses in the joints.

The damage caused by racking could have been probably reduced if the joints were at least strong enough to prevent the tearing of reinforcement out of concrete.

The comparatively heavy damage along axes 3 and 4 is explained by the lack of necessary stiffness in this part of the structure caused by the presence of the stair-well. (This in addition to the fact that this part of the structure was erected slower than the rest of the building). It is suggested that as long as the continuity of floors had to be interrupted by a stair-well, the loss of necessary longitudinal stiffness in this part of the structure could have been compensated by:

1. additional welding of the stair panels to the wall panels and floor slabs;

2. application of reinforcement mesh in the horizontal joints of the longitudinal walls.

Note

Investigation of the above-described case was apparently undertaken by a group of engineers with a view to filling a gap in the 1955 issue of "The Norms and Technical Condition" (NiTU - 127 - 55).

It appears that the norms for the maximum foundation deformation values for precast and large-panel structures are not mentioned in the publication.

Upon completion of their investigation, the group submitted the following recommendations:

1. Soil conditions at the site should be thoroughly investigated; at least 3 test drillings should be made within the outline of the projected structure.

2. In calculating the magnitude of expected foundation settlement in plastic loam, the maximum value of average settlement should be taken as 80 mm (3.15 in.), and the relative maximum value for the difference in settlement as 0.0005.

3. In order to reduce uneveness in the settlement of foundations the reinforced concrete footings should be monolithic and continuous.

4. The panel-reinforcement joints should be stronger than the welded rods connecting 2 adjoining panels (preventing the pulling of reinforcement out of concrete).

5. Connecting rods in floor slabs should be perpendicular to the edges of slabs.

6. The total strength of joints should be the same for any transverse section of the structure.

7. Freezing of soil under foundations should be prevented in the course of construction.

8. Erection of one story should be completed before the work on the next story begun; uneven loading of foundations may thus be prevented.

Source

Stroitel'naya Promyshlennost', No. 5, May 1957, pp. 9-11.

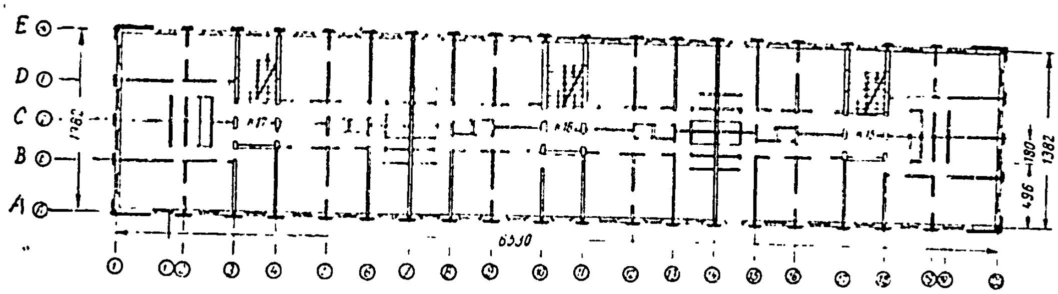

Fig. 1. Plan of the structure

Conversion Table

| cm. | ft. |
|---|---|
| 180 | 5.91 |
| 496 | 16.3 |
| 1,382 | 45.4 |
| 6,530 | 210 |

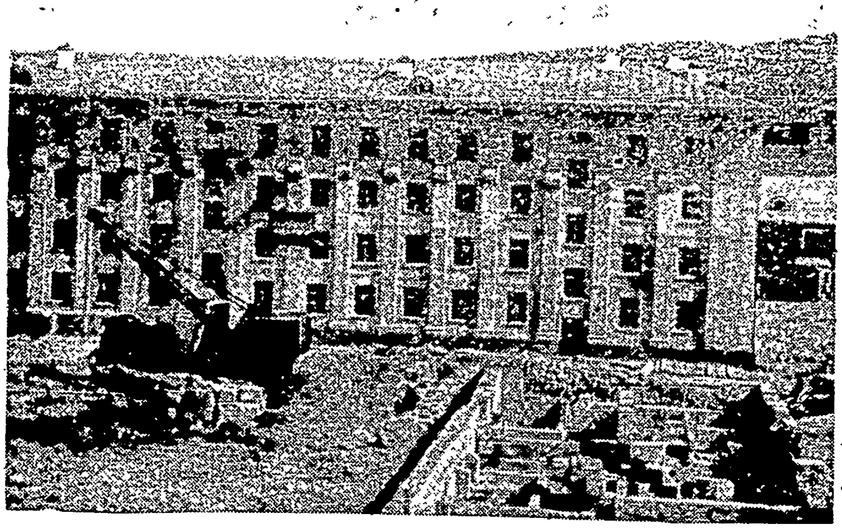

Fig. 2. View of the structure at the end of erection

DAMAGED RESIDENTIAL STRUCTURE AT MURMANSK
Source: Stroitel'naya Promyshlennost', No. 5, May 1957, p. 9

PLATE 14



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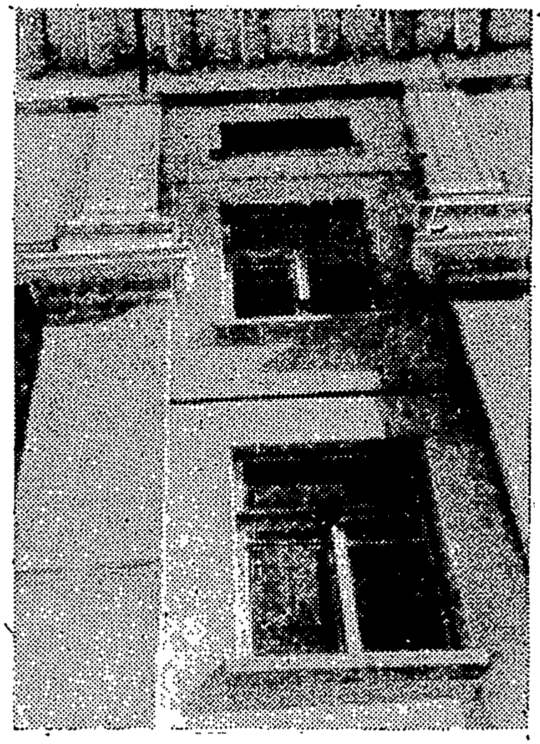

Fig. 1. Crack in the cornice

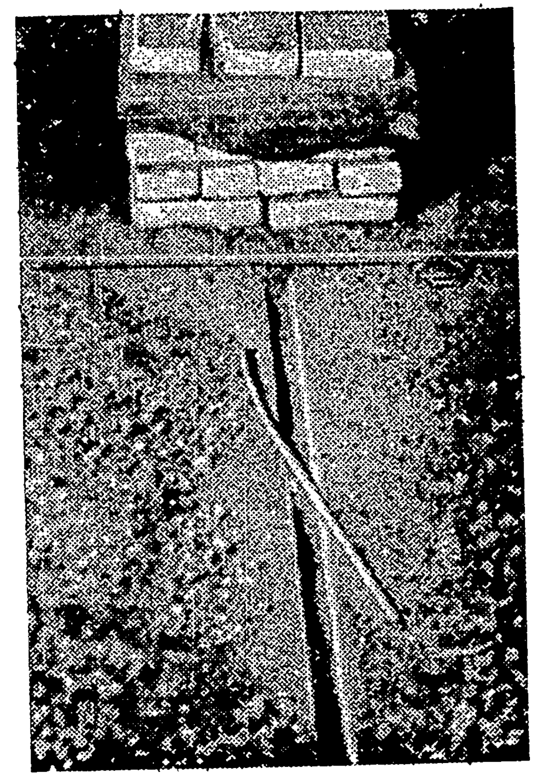

Fig. 2. Separation of slabs in the attic floor

DAMAGED RESIDENTIAL STRUCTURE AT MURMANSK
Source: Stroitel'naya Promyshlennost', No. 5, May 1957, p. 10

PLATE 15



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## CHAPTER VI

### CONDITION OF OLDER CLINKERLESS CEMENT BLOCK BUILDINGS (MIDDLE URAL)

In view of the expansion of the Soviet building industry, Soviet scientists began of late (the source of information as of July 1957) to give serious consideration to the utilization of various locally produced binding substances. The thought apparently is to substitute them for clinker cements in some structures. This led to a restudy of the experience gained with cementless building blocks in the course of the 1930-1934 construction period, as well as to the inspection of buildings of that and later periods which still remain standing.

This inspection was undertaken because, even in the inclement climate of the Middle Ural region, blocks made of local binding substances were sufficiently durable to suggest that the use of this material could be extended.

In many cities of the Ural region, building blocks of 1:2 lime-diatomite composition were widely used in construction between 1930 and 1933; Nizhniy Tagil and Sverdlovsk utilized Kamyshlov diatomites with 75%-78% silica content; Magnitogorsk made use of Troitsk diatomites with 80% -88% silica content.

Data on Lime-Diatomite Blocks

| | |
|---|---|
| Crushing machinery | crusher-roll mill |
| Mixing machinery | mortar mixers; small capacity concrete mixers |
| Aggregate | hardpan, cinders, boiler slag, broken brick, gravel, serpentine, bloated slag, granulated slag. |
| Binding substance ratio | 1:4-4:10 by volume |
| Volume weight of concrete | 1,400-2,200 kg/m$^3$ (87-138 lb/ft$^3$) depending on the kind of aggregate |
| Block curing | Steam -- 2 hours |
| Average Mark of blocks | 25 kg/cm$^2$ (355 lb/in$^2$) - fluctuates from 15 to 35 kg/cm$^2$ (213-497 lb/in$^2$) |
| Frost resistance test | 5-15 freezing cycles |

Photograph of a wooden frame structure with wall filling of the above described blocks (built in 1932) is shown on Plate 16, fig. 1.

Twenty-four such buildings (student dormitories) disintegrated after 2-4 years of occupancy. Incipient wall disintegration (bulging) because of erosion and successive freezing of the lower part of masonry may be seen in photo on Plate 16, fig. 2. Plate 17, fig. 1 shows the state of the buildings after 3-4 years of occupancy.

One building of lime-diatomite blocks in Sverdlovsk lasted 23 years. Three such buildings stand (after a fashion) to this day; one of them is shown in photo on Plate 17, fig. 2. A few blocks taken out of the walls of this building were tested with the following results:

| | |
|---|---|
| Hollow blocks | ultimate strength 6 kg/cm$^2$ (85 lb/in$^2$) |
| Brick aggregate blocks | " " 10-19 kg/cm$^2$ (142-280 lb/in$^2$) |

Also, in Sverdlovsk, 2 three-story frame houses with lime-diatomite block walls are still to be seen. A few years after the erection, the bulging walls of these buildings had to be shored with angle braces as shown in photograph on Plate 18, fig. 1. Especially damaged were the parts of the wall under the windows and around broken drain pipes (Plate 18, fig. 2.). Test blocks taken from the fire wall in the attic had ultimate strength of 21-30 kg/cm$^2$(298-425 lb/in$^2$). Their entire surface, however, was covered with fine setting cracks.

When the diatomite (or any other hydraulic additive) absorbs lime, the initial volume of its particles increases. When blocks are later placed in a dry medium, setting takes place; it causes considerable internal stresses in the blocks and, not infrequently, micro-cracks which seriously reduce the frost-resistant properties of the blocks.

In Nizhniy Tagil, 6 barracks and a school built in 1934 (lime-diatomite blocks with boiler and granulated slag aggregate) had to be demolished as totally unfit for occupancy.

Parts of lime-diatomite block buildings in various stages of disintegration under the influence of outside moisture are shown in photographs on Plate 19, fig. 1 and 2.

A partial view of a lime-diatomite block boiler house is shown on Plate 20, fig. 1 after the blocks in the part of the wall had been replaced with brick and the disintegrated buttresses with logs.

The lime-diatomite blocks may disintegrate even under dry conditions. A part of a crumbling hollow block wall in a dry industrial shop is shown on Plate 20, fig. 2.

A method was found, however, of increasing the durability of these blocks, namely: the addition of portland cement to the lime-diatomite combination. The blocks made by this method were composed of portland cement, lime, diatomite and hardpan in the ratio of 1:1:3:10.

A four-story residential structure was built of large blocks of this type in Sverdlovsk (No. 8 Petrovsky street) during the winter of 1929/1930. Twenty-seven years afterwards, the condition of blocks in the walls was found to be fully satisfactory. No serious signs of disintegration were discovered.

Besides cement-lime-diatomite and/or lime-diatomite block structures, some houses in Nizhniy Tagil were built of lime-cinder blocks which contained some 15-16% of lime. These blocks were of two kinds, with coarse aggregate such as broken brick or boiler slag, or without coarse aggregate. Two 2-story unplastered houses built of those blocks in 1934 are in a comparatively good state.

Houses built in Nizhniy Tagil and Magnitogorsk of large and small lime-slag blocks are in an even better state of preservation.

A corner of a house built of 253 x 79 x 45 cm. (99.6 x 31.1 x 17.7 in.) lime-slag blocks in Magnitorgorsk in 1934 is shown in photograph on Plate 21, fig. 1.

Wall blocks in this and three similar houses are in satisfactory condition; the plinth seems to require some attention.

Lime-slag cement plants were built in Nizhniy Tagil and Magnitogorsk presumably in the early 40's. This led to local manufacture of small building blocks with the following properties:

| | |
|---|---|
| Lime-slag cement | 250-350 kg/$m^3$ (15.7-22 lb/$ft^3$) |
| Average Mark of blocks | 35 kg/$cm^2$ (497 lb/$in^2$) |
| Volume weight | 1,450-1600 kg/$m^3$ (91-100 lb/$ft^3$) |
| Freezing test after manufacturing | 5-7 cycles (Nizhniy Tagilsk)<br>8-12 cycles (Magnitogorsk) |
| Specimen after 3 months in the wall - ultimate strength | 26-62 kg/cm2 (370-880 lb/$in^2$) |
| Freezing test of the above | 10-11 cycles |

Structures built of those blocks in 1945 and later are in satisfactory condition.

Inspection of the above described buildings resulted in the following recommendations:

1. Cementless blocks on lime-diatomite and lime-tripolite base, such as were used in the Ural region, are insufficiently frost resistant and durable. They should not be used in construction of outside walls.

2. Small and large blocks on lime-diatomite (tripolite) base with addition of Portland cement are sufficiently durable. They may be used in outside wall construction, provided:

   a. Mark of blocks is not lower than 35 kg/$cm^2$;
   b. Block frost resistance is not lower than 10 standard cycles;
   c. The plinth blocks are of a more frost resistant material and are properly waterproofed;
   d. The overhang of the eaves is increased (Plate 21, fig. 2)
   e. The window openings are faced with frost resistant material.

3. Lime-cinder blocks (cinder composition based on laboratory and factory tests) are more durable than those of lime-diatomite. Manufactured with the addition of 70 kg/$m^3$ of the clinker cement, such blocks may be used in outside walls of buildings which are not higher than 3 stories. They must comply with conditions of paragraph 2 above.

4. Small and large blocks on lime-granulated blast furnace slag base form a fully satisfactory wall material which withstood the 20-25 year service test under relatively severe conditions. It may be recommended as wall material for substantial structures.

In the manufacture of lime-slag cement, activated granulated blast furnace slag should be used. Its specific gravity should not exceed 0.9, its manganese content should be less than 3%, its adhesive value should not be lower than 150 kg/cm2 (2130 lb/$in^2$).

Composition of light concrete should be so selected that the strength of solid blocks is at least 50 $kg/cm^2$ (710 $lb/in^2$), and of hollow blocks not less than 35 $kg/cm^2$ (497 $lb/in^2$).

Note:

It may be assumed on the strength of the above study that greater use of lime-slag blocks in the Soviet building industry is indicated.

Source

Stroitel'naya Promyshlennost', No. 7, July 1957, pp. 2-7

Fig. 1. Residential structure with wood frame and lime-diatomite block filled walls (built in 1932).

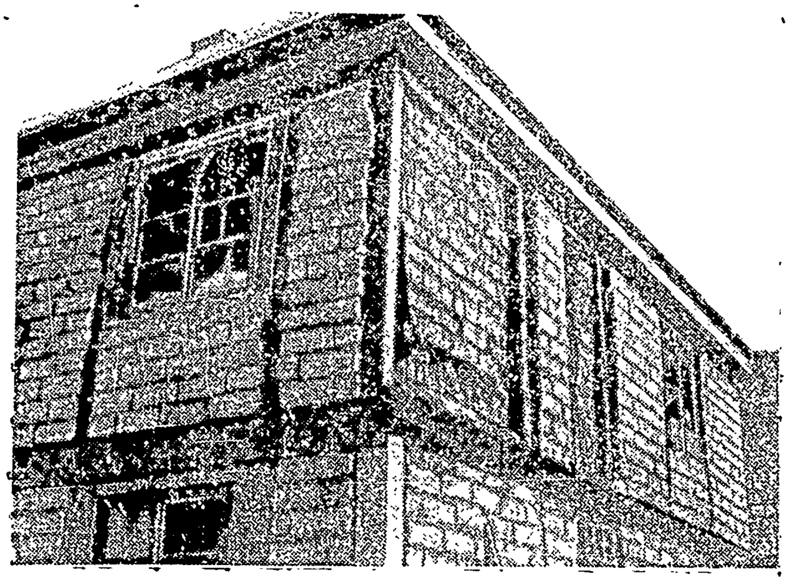

Fig. 2. Incipient disintegration of lime-diatomite block walls 2-4 years after erection.

LIME-DIATOMITE BLOCK STRUCTURES IN THE MIDDLE URAL REGION
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 3

PLATE 16

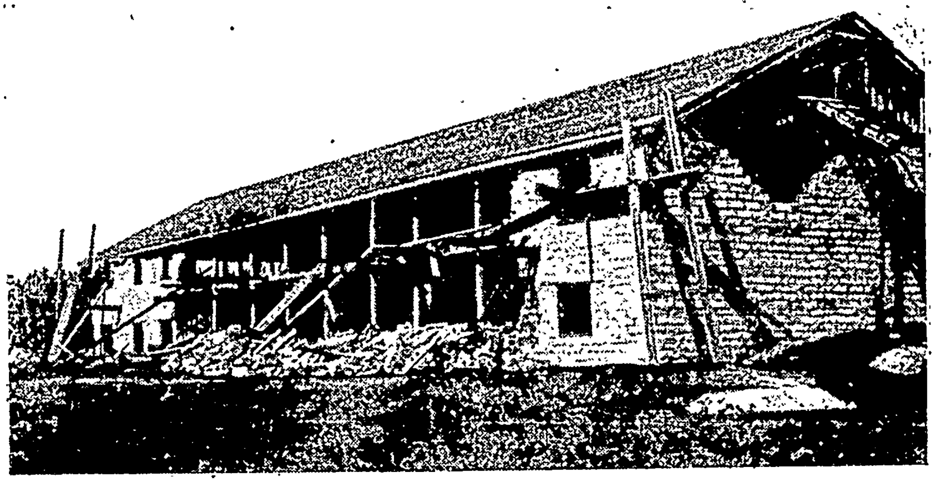

Fig. 1. Disintegration of the structure 3-4 years after erection

Fig. 2. Lime-diatomite block structure in Sverdlovsk 25 years after erection (strength of hollow blocks: 85 $lb/in^2$)

LIME-DIATOMITE BLOCK STRUCTURES IN THE MIDDLE URAL REGION
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 3

PLATE 17

Fig. 1. Outside appearance of a 3-story lime-diatomite structure at Sverdlovsk 25 years after erection (the walls were shored a few years after erection)

Fig. 2. Disintegration of the walls under the windows and around the drain pipe (strength of the blocks in the fire wall: 298-425 lb/in2)

LIME-DIATOMITE BLOCK STRUCTURES IN THE MIDDLE URAL REGION
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 4

PLATE 18

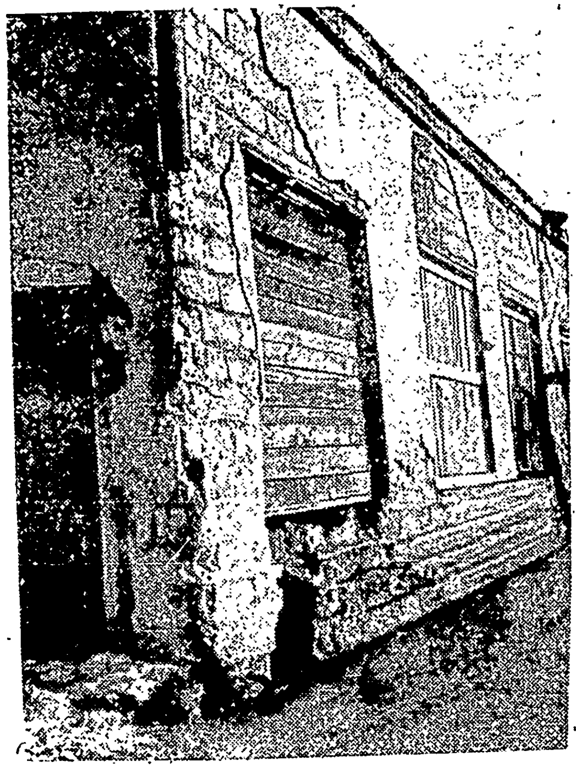

Fig. 1. Disintegration of a corner of a store caused by excessive moisture; (part of a broken drain pipe appears at the upper left corner)

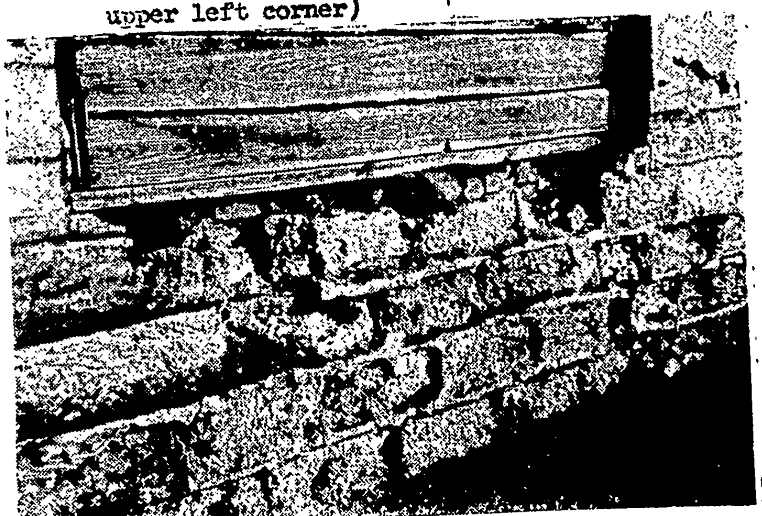

Fig. 2. Disintegration of blocks under window

LIME-DIATOMITE BLOCK STRUCTURES IN THE MIDDLE URAL REGION
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 5 & 6

PLATE 19

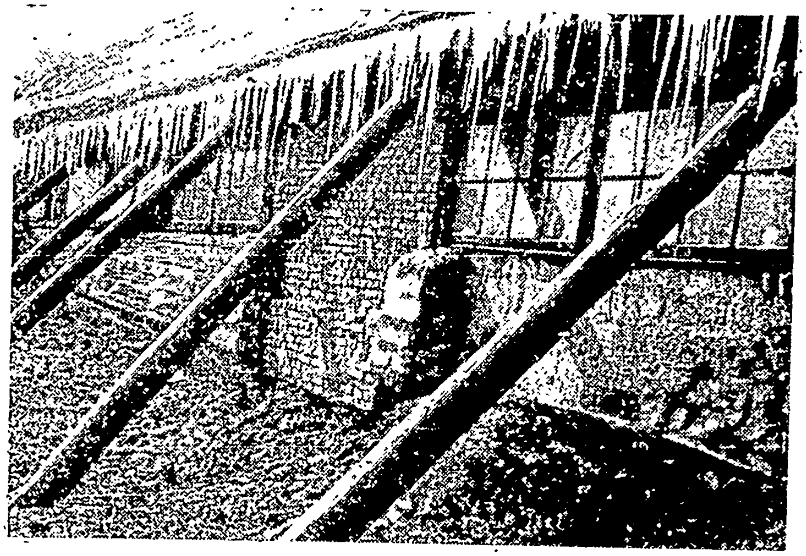

Fig. 1. Disintegration of a wall in a boiler house; (part of the block wall was replaced with brick; logs took place of disintegrated buttresses)

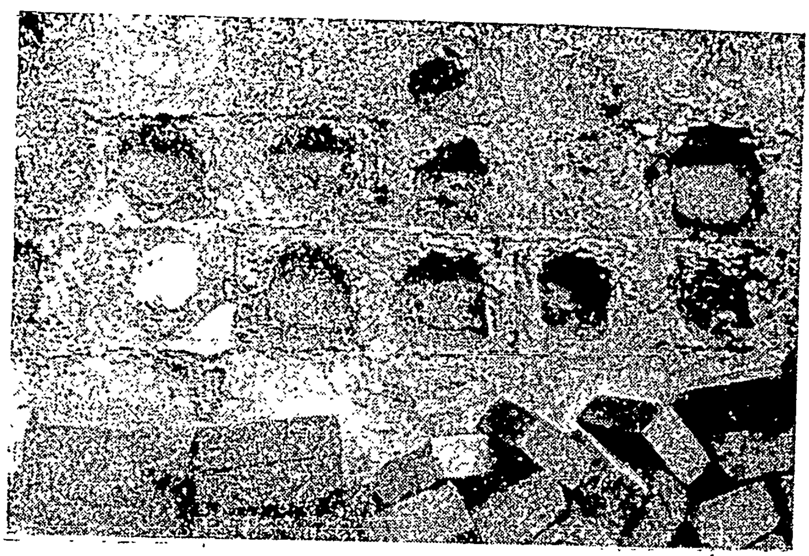

Fig. 2. Disintegration of the inside face of a wall in a dry industrial shop. (Shop operational conditions could have contributed to the damage).

LIME-DIATOMITE BLOCK STRUCTURES IN THE MIDDLE URAL REGION
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 6
PLATE 20

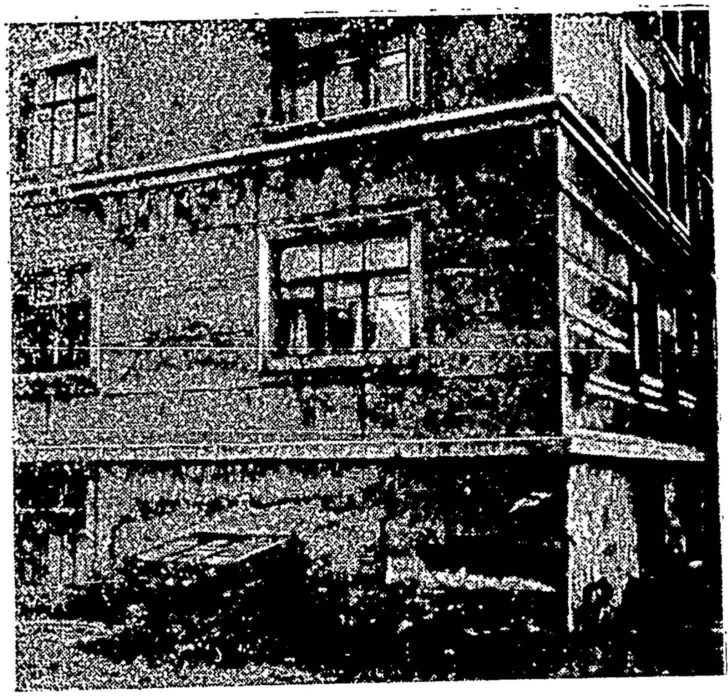

Fig. 1. Corner of a house built at Magnitogorsk in 1934 of large lime-slag blocks. Condition of the structure except the plinth is stated to be satisfactory.

Fig. 2. Disintegration of a lime-diatomite block wall caused by weathering. The narrow eaves provide no protection from moisture.

LIME-SLAG AND LIME-DIATOMITE STRUCTURES IN THE MIDDLE URAL REGION
Source: Stroitel'naya Promyshlennost', No. 7, 1957, pp. 5-6
PLATE 21



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# CHAPTER VII

## DEFECTIVE CONSTRUCTION IN WHITE RUSSIA

A signed article in a recent (March, 1957) Soviet building trade magazine indicates that faulty construction is widespread in White Russia.

The names of the building organizations, officials and engineers responsible for the poor work are given in the article. Further substantiation is provided by four photographs of defective structures (Plate 22, figs. 1-4) and the following additional facts:

1. During the construction of a publishing house for the Central Committee of the White Russian Communist Party at Minsk:

   a. reinforced concrete lintels of inadequate length were used;
   b. reinforced concrete floor slabs (presumably precast) were of inferior quality;
   c. the brick used was of such poor quality that load-bearing piers collapsed under partial load.

2. In a 156- apartment residential structure on Stalin Prospect (Stalin Street) at Minsk, the collapse of a combined ventilation-smoke uptake led to the collapse of floor slabs, presumably on several stories.

3. In a residential structure housing the workers of the Minsk watch factory, five floors collapsed because of the low quality of wall construction and faulty joints of panels with their supports.

4. Similar mishaps occurred at other building sites in Minsk.

5. In 1956, at Minsk, tenants had moved into 16 residential structures before the construction was completed and the building accepted by the State Commission. The occupancy was authorized by the Minsk City Executive Committee whose duty it was to combat such practices.

6. At Vitebsk, a case of structural failure took place.

7. At Mogilev, the finishing work in a series of residential structures was unsatisfactory.

The article ends with the following challenging words:

> "The editorial office awaits information from the Ministry of Urban and Rural Construction as well as from the Ministry of Construction of the White Russian Soviet Socialistic Republic concerning the measures adopted with a view to improving the quality of construction work so that this information can be published in this magazine".

Source

Stroitel', No. 3, March 1957, p. 13

Fig. 1. Reinforced concrete floor slabs after thaw. Wall may be seen through one of the holes

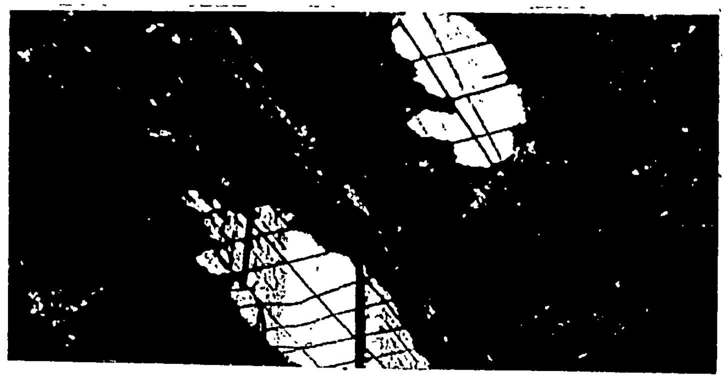

Fig. 2. Faulty concrete work

Fig. 3. Faulty brickwork

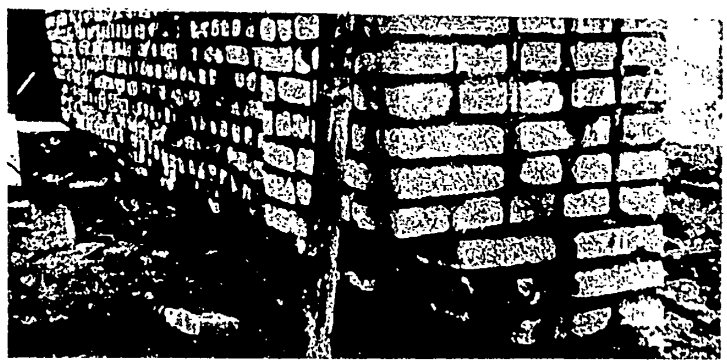

Fig. 4. Reinforced concrete panel with disintegrating load-bearing rib

EXAMPLES OF DEFECTIVE CONSTRUCTION IN WHITE RUSSIA
Source: Stroitel', No. 3, March 1957, p. 13
PLATE 22



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# CHAPTER VIII

## DEFECTIVE RESIDENTIAL STRUCTURES IN A WORKERS' VILLAGE BY THE NOVY-GOR'KI REFINERY

There is no doubt that the Soviet authorities are dissatisfied with current Soviet building standards.

Whatever other methods may be employed to improve these standards, one of the methods appears to be giving widespread adverse publicity to hapless builders and their faulty works.

Thus, under the general heading: "The Facts Accuse The Bradkodiely (the rejects makers)", a trade magazine published, in October 1956, a page of 7 photographs of defective structures with rather caustic captions. The editors did not apparently think that an explanatory text was needed to accompany them.

Four photographs "take to task" Trust No. 114 (and its administrative chief) of the Construction Ministry for the Petroleum Industry Enterprises of the USSR by depicting structural defects in residences of workers' village by the Novy Gor'ki refinery. These photographs are shown on Plates 23 and 24.

Two photographs provide examples of poor construction work by the "Krivorozhstroi" Trust; one photograph gives a view of a partially collapsed industrial structure erected by the "Kadievshakhstroi" Trust.

Strictly speaking, the last three photographs do not belong in this particular chapter. Nevertheless, in order to preserve the unity of the original material they are reproduced on Plate 25.

<u>Source</u>

Stroitel', No. 10, October 1956, p. 22

Fig. I. "Two years after the housewarming the tenants had to be moved out. The house was disintegrating."

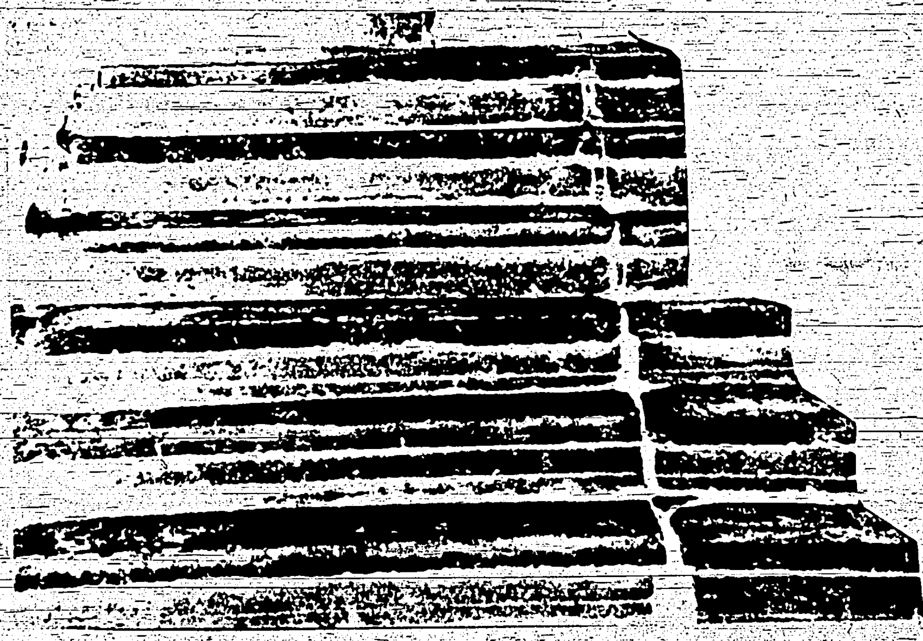

Fig. 2. "Has I. P. Silenko (administrative chief of building trust No. 114) seen these concrete steps in house No. 5?"

DEFECTIVE RESIDENTIAL STRUCTURES IN A WORKERS' VILLAGE BY THE NOVY GOR'KIY REFINERY

Source: Stroitel', No. 10, October 1956, p. 22

PLATE 23

Fig. 1. "House No. 18 in block 3 was built by Trust No. 114. Defects of such brickwork could only be hidden under a thick layer of plaster."

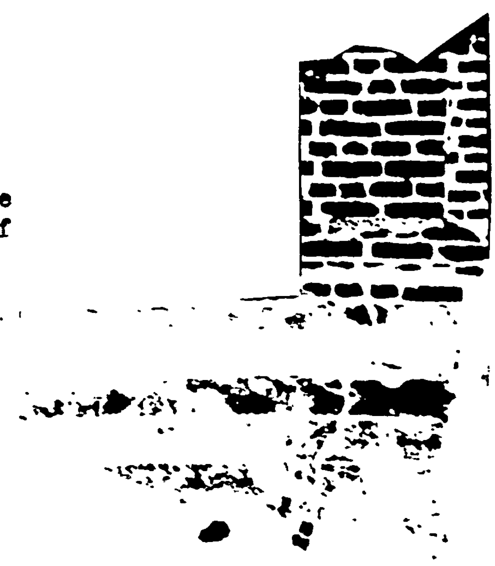

Fig. 2. "Cracks appeared in the concrete plinth and brickwork because of the settlement of foundations. (Built by Trust No. 114)."

DEFECTIVE RESIDENTIAL STRUCTURES IN A WORKERS' VILLAGE BY THE NOVY-GOR'KIY REFINERY

Source: Stroitel', No. 10, October 1956, p. 22

PLATE 24



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Display heading: "Facts Accuse the Rejects Makers".

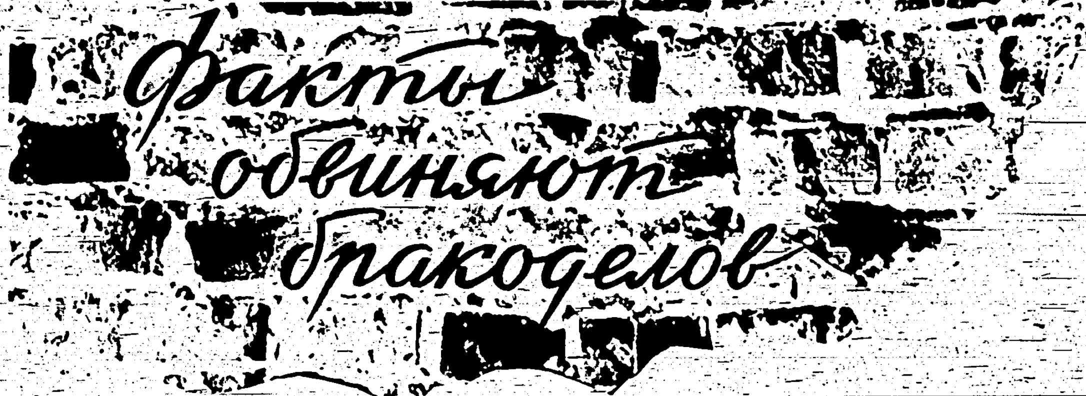



Fig. 1. "This brickwork was built by 'Krivorozhstroy' Trust".

Fig. 2. "The collapse of loading bunkers, trestles and reloading stations at Krinichanskaya-Severnaya mine in Donbass was due to the poor quality of concrete in columns; (built by 'Kadievshakhstroy' Trust)".

Fig. 3. "This is what happens when the door frame is anchored improperly. (Built by the 'Krivorozhstroy' Trust)".

EXAMPLES OF DEFECTIVE BUILDING BY "KRIVOROZHSTROY" AND "KADIEVSHAKHSTROY" TRUSTS
Source: Stroitel', No. 10, October 1956, p. 22

PLATE 25

# PART TWO

# INDUSTRIAL STRUCTURES

# CHAPTER IX

## COLLAPSE OF A DOUBLE CURVED ROOF IN A TEXTILE PLANT (MINSK)

### Foreword

This account is prepared on the basis of information gained from two sources. Although one of them does not give the location of the plant in question, it is assumed that both sources refer to the same case because:

a) the type of the structure and certain given dimensions are the same;
b) the year and the month of the failure are also the same.

### Location

Textile Trust Plant, Minsk, White Russia.

### Structure

One-story reinforced concrete structure with double-curved reinforced concrete shell roof. It is the main building of the plant housing carding, spinning and weaving shops.

### Construction

Plan of the structure is shown on Plate 26; longitudinal and transverse sections on Plates 26, 27, and 28; photographs of the plant in the process of construction are given on Plates 29 and 30.

The structure has 12 aisles separated by four corridors and an expansion joint (along axes 17 and 18, plan, Plate 26). There are no transverse expansion joints.

Dimensions:

| | | |
|---|---|---|
| Overall width | | - 287.63 m (945 ft.) |
| Overall length | | - 201.00 m (660 ft.) |
| overall height | approx. | - 12.00 m ( 39.4 ft.) |
| Width of corridors | | - 5.00 m ( 16.4 ft.) |
| Width of corridor along axes 7-9 (Plate 26) | | - 10.00 m ( 32.8 ft.) |
| Width of expansion joint | | - 0.63 m ( 2.07 ft.) |
| Aisle width | | - 21.00 m ( 68.9 ft.) |
| Bay length | | - 12.00 m ( 39.4 ft.) |

Roof: The roof is built in sections composed of:

a) Flat slabs which cover the corridors;

b) Two monolithically joined double-curved shells which cover each pair of adjacent aisles.

The flat roof slabs of the corridors are of precast reinforced concrete.

The shell roof over the aisles is constructed as follows:

1. Each individual shell has the shape of a part of ellipsoid forming the surface of positive gaussian curvature. If this surface is intersected in its upper and lower parts by vertical planes we shall obtain the outlines of the arch and of the more gently sloping lower edge of the roof shell under consideration. The slope of the shell with respect to the plane of the floor is approximately 30°.

2. The shell is "flanged" by four curvilinear members which are joined with it monolithically. These members are:

a) The arch; the arch-shell joint is shown on Plate 28, fig. a.

b) The side member; its cross-section is 200 X 650 mm (7.87 X 25.6 in.); it is provided with an 80 mm (3.15 in.) thick and 600 mm. (23.6 in.) high dwarf wall which supports the flat slabs of the corridor roof; the side member-shell joint is shown on Plate 27, fig. b.

c) The middle member; its cross-section is 200 X 700 mm. (7.87 X 27.6 in.); it joins monolithically two adjacent shells; the middle member-shell joint is shown on Plate 27, fig. c.

d) The lower member; this member forms the tie beam of the next arch in the longitudinal direction, to which it is connected by means of four steel brick-faced suspension supports (Plate 27); the tie beam, suspension supports and the arch provide support for the window frames; the lower member-shell joint is shown on Plate 28, fig. d.

3. The shell and its members are of Mark 200 reinforced concrete on Mark 500 cement. The size of aggregate is about 15-20 mm. (0.59-0.787 in.). The shell is 5 cm. (1.97 in.) thick at the center, 10 cm. (3.94 in.) at the arch and the tie beam and 20 cm. (7.87 in.) at the side and middle members.

4. The shell reinforcement consists of a single row of 8 mm. (0.315 in.) steel rods spaced at 100 mm. (3.94 in.) in the middle of the shell and of a double row at the edges. The rods are laid individually and fastened in the concrete form of the shell. Shell reinforcement distribution is indicated on Plate 27, figs. b and c and Plate 28, figs. a and d.

5. Construction of the shells was facilitated by the use of a specially built movable concrete form which permitted the concreting of an entire shell (an area of 252 $m^2$ or 2,700 $ft^2$) from a single position. On the other hand, the use of the form presented a serious drawback, namely: the tie beam could not be poured simultaneously with the columns for it would create an obstacle to the rolling of the form into the next bay; the reinforcement could be placed and the tie beam poured only in conjunction with the pouring of the next shell. The form, in its lowered state, could be moved out only under the 12 m.(39.4 ft.) side member for placing in the next bay. A photograph of such a form is shown on Plate 31.

(Note: It was found necessary in the course of construction to strengthen the tie beams. The reason is explained under "Construction changes and time table" below.)

Columns. The 600 X 400 mm.(23.6 X 15.7 in.) reinforced concrete columns are poured in place.

Column joints. The columns are joined with the tie beams, arches and side members of the shells by means of "Perederiy type" joints (so named after the Soviet academician, G. P. Perederiy, who had designed them).

Perederiy joints are lap joints made without welding with reinforcement loops and steel joint tongues. They were devised for precast reinforced concrete construction in the early days of development of that technique. With wider application of welding to reinforcement, the Perederiy joint went out of use. Column-tie beam joints of Perederiy type are shown in drawings on Plate 32, figs. 1-4.

Construction changes and time table. Construction was started some time in 1952. By March 1956, the construction progressed to a point where, roughly, out of 630,000 $ft.^2$ of the building's floor space some

150,000 $ft.^2$ were already assigned to actual production operations;
120,000 $ft.^2$ had the textile equipment in the process of installation;
40,000 $ft.^2$ were undergoing finishing work;
320,000 $ft.^2$ had only the roof and the walls completed.

Two last roof shells between axes 12-14 (plan, Plate 26) were completed with the aid of steam heating in December, 1953. The heating proved ineffective and concrete inadequate. The shells were dismantled and re-erected in March, 1954. This part of the structure remained without heating or protective roof covering for almost 2 years, heat insulation being applied only in the winter of 1955/56 when cold weather with temperatures between -30 and -35° C (between 22 and 31°F below 0) prevailed.

As early as 1954, cracks, tapering from the top down, were noted in some column-tie beam joints (Plate 32, fig. 2). To strengthen the roof and to forestall the appearance of similar cracks in other joints, the tie beams were strengthened with paired, two-aisle-long steel channels No. 16 (no weight given; see SES Report No. 1, Table 1.0253B). The channels were welded at the ends with steel channel No. 20 scrap pieces; steel wedges were driven between these pieces and the column (Plate 33, fig. 1). Thus, the paired channels began to serve as additional ties to the arched members of the roof. Such channels were erected along all the tie beams with Perederiy joints.

The Perederiy joint reinforcement projections were welded thereafter.

## Partial Collapse of the Structure

Sometime in March 1956, approximately 4,500 $m^2$ (48,500 $ft^2$) of the as yet non-operational part of the building collapsed.

The affected part (it had Perederiy lap joints and strengthening steel channels) is shown by shaded area in plan on Plate 26.

Photographs of the wreckage are shown on Plates 34 and 35.

The failure probably began with the rupture of reinforcement in one of the structural members. It developed as follows. First, two last shells between axes 12 and 13 collapsed; in a few minutes, they were followed by two adjoining shells between axes 13 and 14; in about half an hour, 18 shells between axes 12 and 14 went down. The failure stopped at a transverse 3-stretcher brick partition wall with pilasters, laid in the plane of the arch. The development of the failure is illustrated by sketches on Plate 36, figs. 1-3.

## Causes of Failure

The collapse was due to an interplay of many factors, the chief among them being the following:

1. One week before the collapse, the part where the failure began was temporarily partitioned off. This was done with the view to completing the finishing work and installing equipment, all of which required the heating of the place. The heating arrangement was simple. Two steam pipe heaters were suspended in the immediate vicinity of the internal steel channels which originally were supposed to strengthen the resistance of the tie-beams to tension. The outside channel remained cold. The arrangement is shown on Plate 33, figs. 2 and 3.

2. The cracks in the joints which were noticed in 1954 and the fact, unnoticed at the time, that the columns were out of plumb indicated that there were shifts in Perederiy joints. The construction of the joints was found, in general, to be faulty. In some joints, the 10 mm.(0.394 in.) steel joint tongues were shifted; in the other, they were missing altogether; in still others the reinforcement overlap was insufficient.

3. The cracks similar to those in the column-tie beam joints were also noticed in the column-shell side member joints. Their occurrence may not have been altogether due to the faulty assembly of reinforcement in the Perederiy joints. The use of the movable concrete form unavoidably produced "between-the-pours" joints in concrete where columns were joined with other structural members. Thus, truly monolithic joints could not be achieved in some instances. Hence the cracks.

4. Steel reinforcement in bearing members was found to be very brittle. Laboratory tests of steel specimens indicated:

a) carbon content of the steel was 0.33-0.4%;
b) steel contained considerable amount of slag;
c) relative elongation of steel was 5.7%.

Some reinforcing rods were cracked even before they were placed in concrete forms; the joining of vertical and horizontal reinforcement rods was done not by the spot-welding but by the less satisfactory arc-welding method.

5. Layers of ice some 1.7 m.(5.6 ft.) thick and weighing from 10 to 12 tons formed in roof valleys because of alternate freezing and thawing of snow and rain.

6. The lack of stiffness in the structural system as a whole and in the arch-tie beam elements in particular was probably responsible for the peculiar chain pattern which the failure assumed. On the other hand, this chain pattern tendency seems to be inherent in the very design of the roof, for each arch-tie beam element is the main load-bearing member for two successive shells; the preceding shell rests on the arch, the next shell on the tie beam of that arch.

## Reconstruction Project

Reconstruction, actually, presented two problems, namely:

a) strengthening of the still standing parts of the structure;
b) rebuilding of the part that had collapsed.

Strengthening of the Structure. It was proposed that the following steps be taken to strengthen the part of the structure still standing:

1. Arch-tie beam elements were to be stiffened in their upper parts by means of paired steel channels in a manner similar to that employed in 1954 when cracks in column-tie beam joints were first noticed (Plate 37, figs. 1 and 2). The channels were to be encased in light concrete for the purpose of protecting them from temperature variations. No heaters were to be placed near them.

2. The arch-tie beam element columns separated by 2 or 4 bays were to be encased to their full height in concrete in accordance with the drawings shown on Plate 37, figs. 1-6.

3. The idea of separating the arch from the tie beam by cutting the four suspension supports was considered for the arch-tie beam elements resting on strengthened columns. The idea was abandoned because the tie beam cross section was found to be too small. Two stiffening, two-aisle-long steel channels were used instead (Plate 37, fig. 1 and 2).

4. The expansion joint columns (plan, Plate 26) were to be joined into single solid columns; the expansion joint was to be eliminated after the heating of the reconstructed building would preclude the possibility of sharp temperature fluctuations.

Rebuilding the Part that Collapsed. It was recommended that the rebuilding of the 18 bays (9 in each of the two affected aisles) that had collapsed follow in general the original plan of construction with the following slight modifications:

a) the arch-tie beam elements are to be joined with columns by means of more reliable stiff reinforcement;

b) reinforcement in column-shell side member joints should be strengthened.

## Note

Construction of large double-curved roof buildings for housing light industries was started in the Soviet Union apparently in 1950. In addition to the above described plant at Minsk, similar structures were presumably erected at Kalinin and Vyshniy Volochek.

## Source

1) Stroitel'naya Promyshlennost', No. 2, 1957, pp. 21-26.
2) Stroitel'naya Promyshlennost', No. 8, 1955, pp. 9-10.
3) Moscow TsINIS. Causes of Structural Failures, pp. 49-58; TH 3401.M7

Conversion Table

| mm. | ft. |
|---|---|
| 400 | 1.31 |
| 500 | 1.64 |
| 600 | 1.97 |
| 630 | 2.07 |
| 2,010 | 6.60 |
| 2,100 | 6.90 |
| 2,400 | 7.87 |
| 3,000 | 9.84 |
| 4,500 | 14.8 |
| 4,570 | 15.0 |
| 5,000 | 16.4 |
| 5,800 | 19.0 |
| 6,580 | 21.6 |
| 10,000 | 32.8 |
| 12,000 | 39.4 |
| 21,000 | 69.0 |
| 237,630 | 944. |

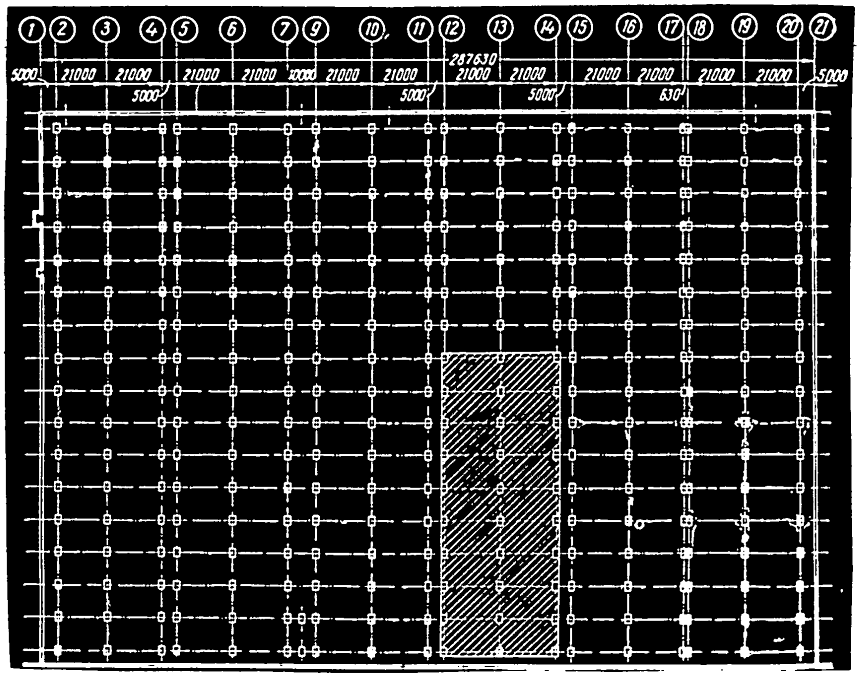


Fig. 1. Plan of the plant. Shaded area indicates the part that collapsed.

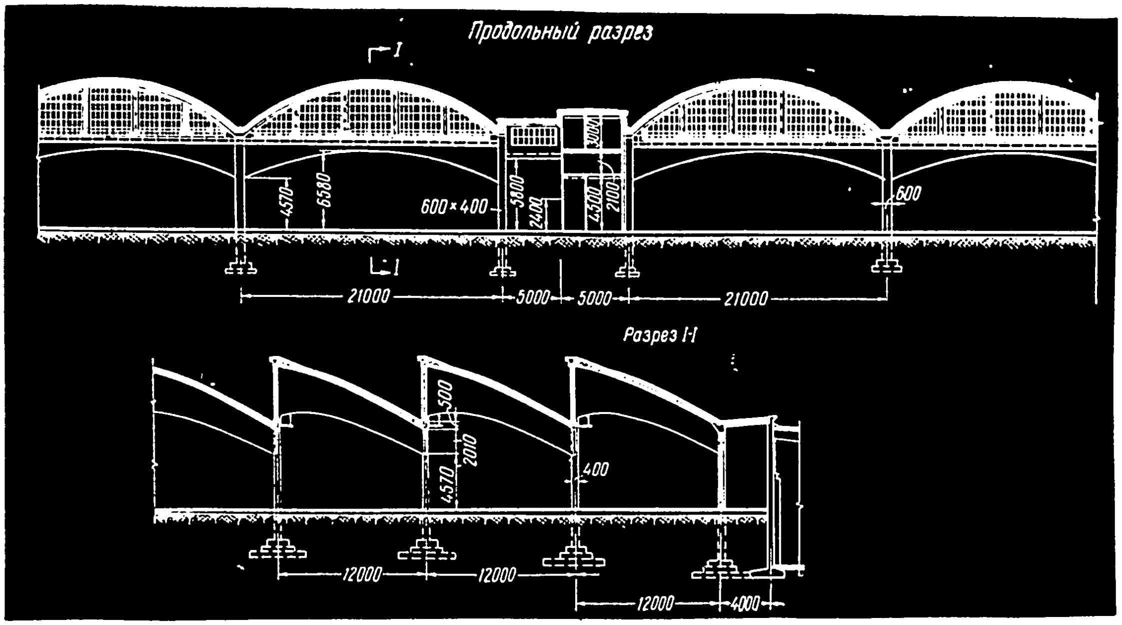
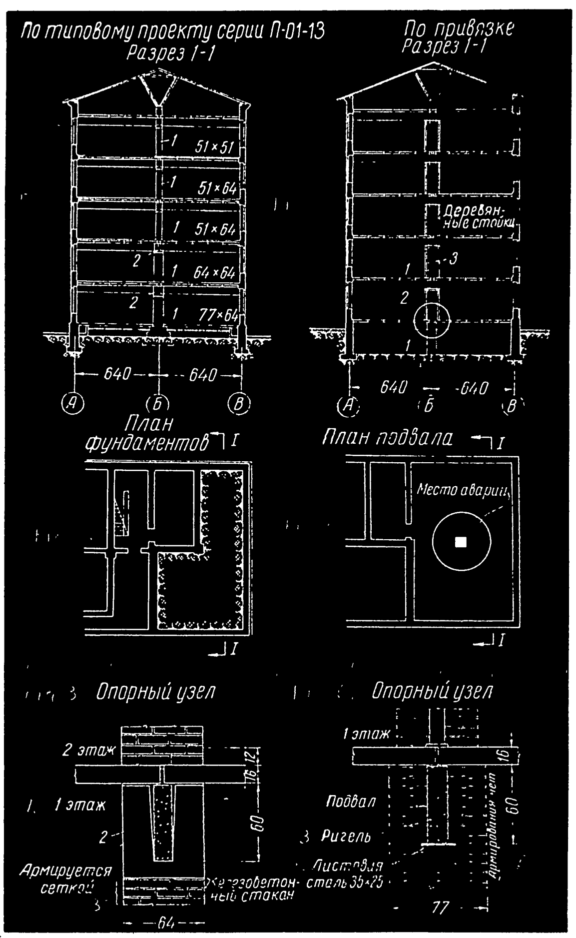



Fig. 2. Transverse and longitudinal sections

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Moscow TsINIS. Causes of Structural Failures, pp. 51-52. (TH 3401.M7)

PLATE 26




Figs. 1,2,3 - Structure as designed; Figs. 4,5,6 - Structure as actually built.
Sections and details of the structure
The collapsed pier is marked with a circle.

RESIDENCE No. 16 CHAPLYGIN STREET, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 14.
(TH 3401.M7)

PLATE 7





## Design

Fig. 1. Section 1-1
1. Brick piers
2. RC beam socket

Conversion Table

| cm. | in. |
|---|---|
| 51 | 20.1 |
| 64 | 25.2 |
| 77 | 30.3 |
| 640 | 252.0 |

Fig. 2. Plan of Foundations

Fig. 3. Joint (pier-beam-floor slabs)
1. First floor
2. RC beam socket
3. Mesh reinforced pier

Conversion Table

| cm. | in. |
|---|---|
| 12 | 4.72 |
| 16 | 6.30 |
| 60 | 23.6 |

## Actual Construction

Fig. 4. Section 1-1
1. Brick piers
2. Joint (pier-beam-slab)
3. Wooden posts

Fig. 5. Plan of the cellar

Fig. 6. Joint (pier-beam-floor slabs)
1. First floor;
2. Cellar
3. Beam
4. Steel bearing pad (13.8 x 9.4 in.)
5. Pier without reinforcement

Conversion Table

| cm. | in. |
|---|---|
| 16 | 6.30 |
| 60 | 23.6 |
| 77 | 30.3 |

Fig. b. Side member-shell joint

Fig. c. Middle-member-shell joint

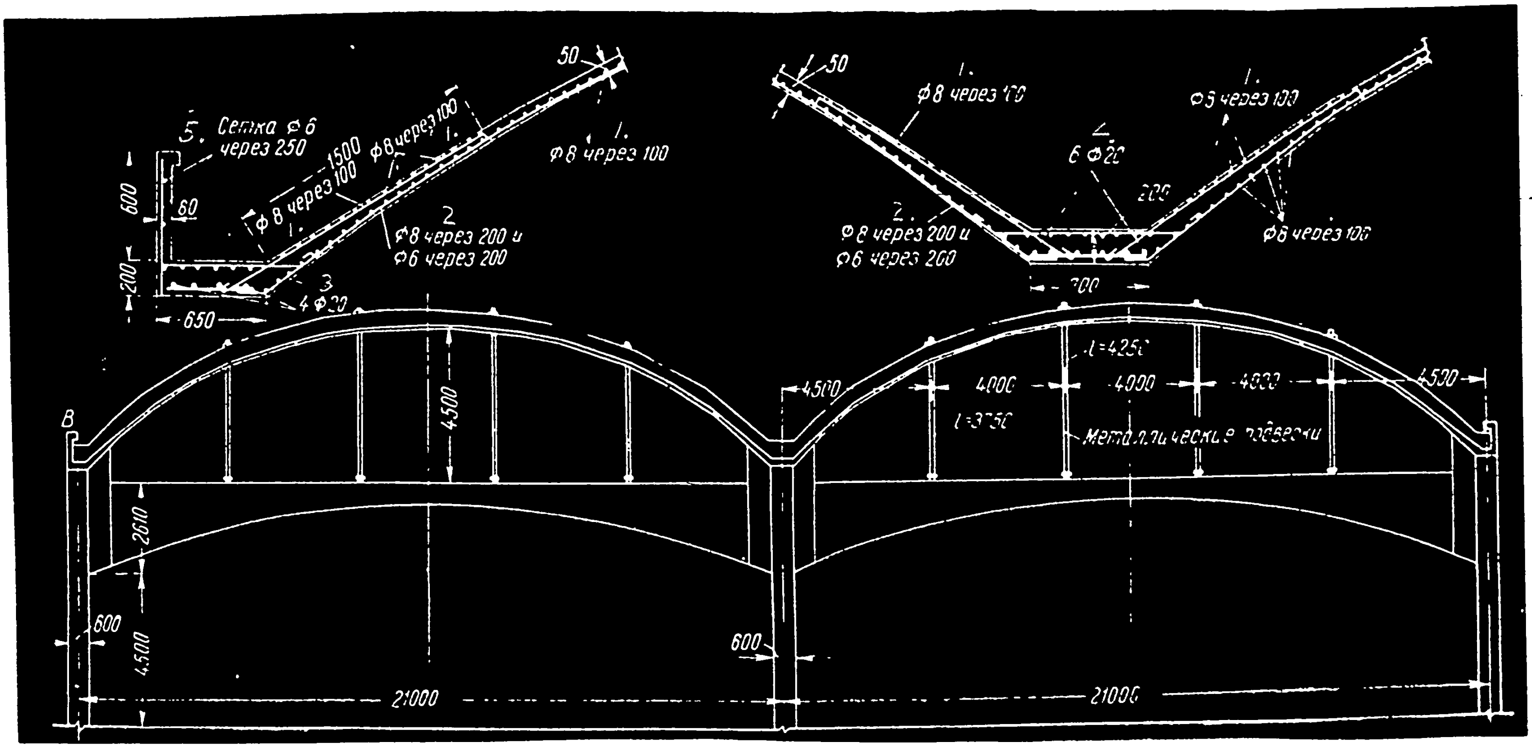



Fig. a. Transverse section

Transverse Section and Side Member-Shell and Middle Member-Shell Joints

Conversion Table

| mm. | in. | mm. | ft. |
|---|---|---|---|
| 6 | 0.236 | 1,500 | 4.92 |
| 8 | 0.315 | 2,610 | 8.57 |
| 20 | 0.787 | 3,750 | 12.3 |
| 50 | 1.97 | 4,000 | 13.1 |
| 100 | 3.94 | 4,250 | 13.9 |
| 200 | 7.87 | 4,500 | 14.8 |
| 250 | 9.84 | 21,000 | 68.9 |
| 600 | 23.6 | | |
| 650 | 25.6 | | |
| 700 | 27.6 | | |

Reinforcement

1. d = 0.315 in.; spacing = 3.94 in.
2. d = 0.315 in.; spacing = [illegible].87 in.
   d = 0.236 in.; spacing = 7.87 in
3. 4 rods, d = 0.787 in.
4. 6 rods, d = 0.787 in.
5. Mesh: d = 0.236 in.; spacing = 9.84 in.

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Stroitel'naya Promyshlennost', No. 2, 1957, p. 22

PLATE 27

POOR ORIGINAL

Longitudinal Section

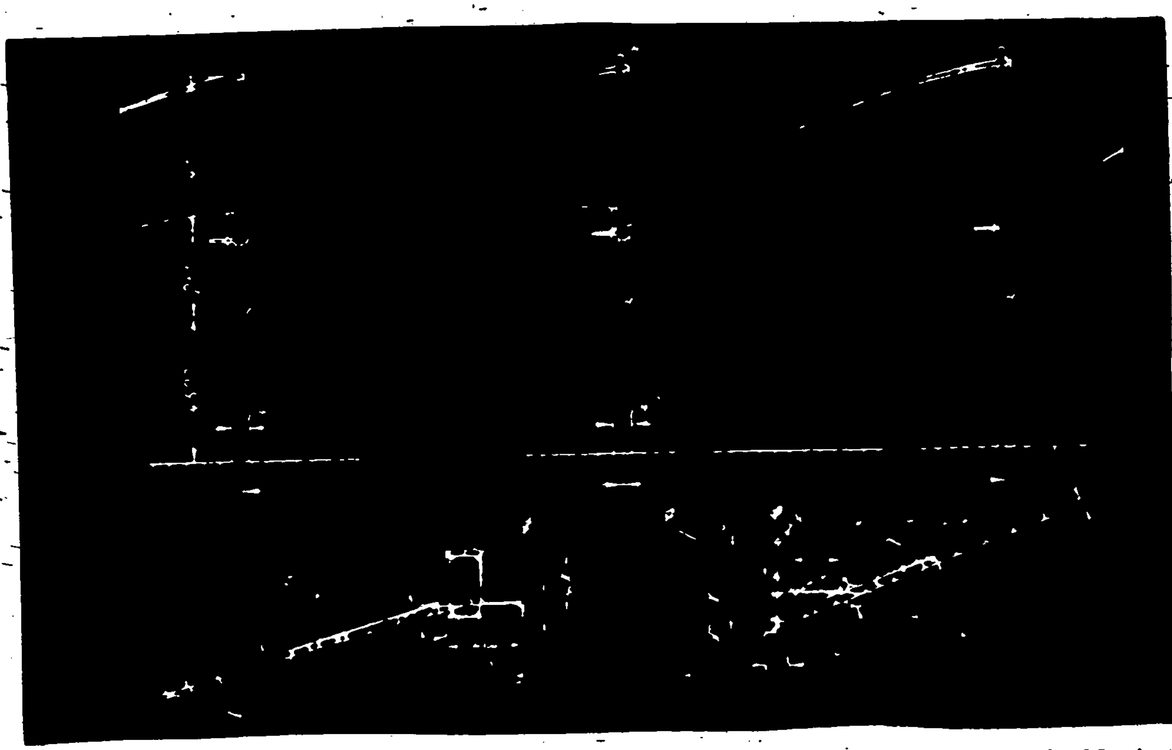


Fig. a. Arch-shell joint

Fig. d. Lower member-shell joint

1. Mesh: d=0.236 in.; spacing = 7.87 in.
2. Two rods, d=0.394 in.
3. Two rods, d=0.787 in.
4. Two rods, d=1.18 in.

Conversion Table

| mm. | in. | mm. | in. | mm. | ft. |
|---|---|---|---|---|---|
| 6 | 0.236 | 150 | 5.91 | 2,610 | 8.56 |
| 10 | 0.394 | 200 | 7.87 | 4,500 | 14.8 |
| 20 | 0.787 | 250 | 9.84 | 12,000 | 39.4 |
| 30 | 1.18 | 260 | 10.2 | | |
| 50 | 1.97 | 310 | 12.2 | | |
| 80 | 3.15 | 400 | 15.7 | | |
| 100 | 3.94 | 1,000 | 39.4 | | |

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK

Source: Stroitel'naya Promyshlennost', No. 2, 1957, p. 22

PLATE 28



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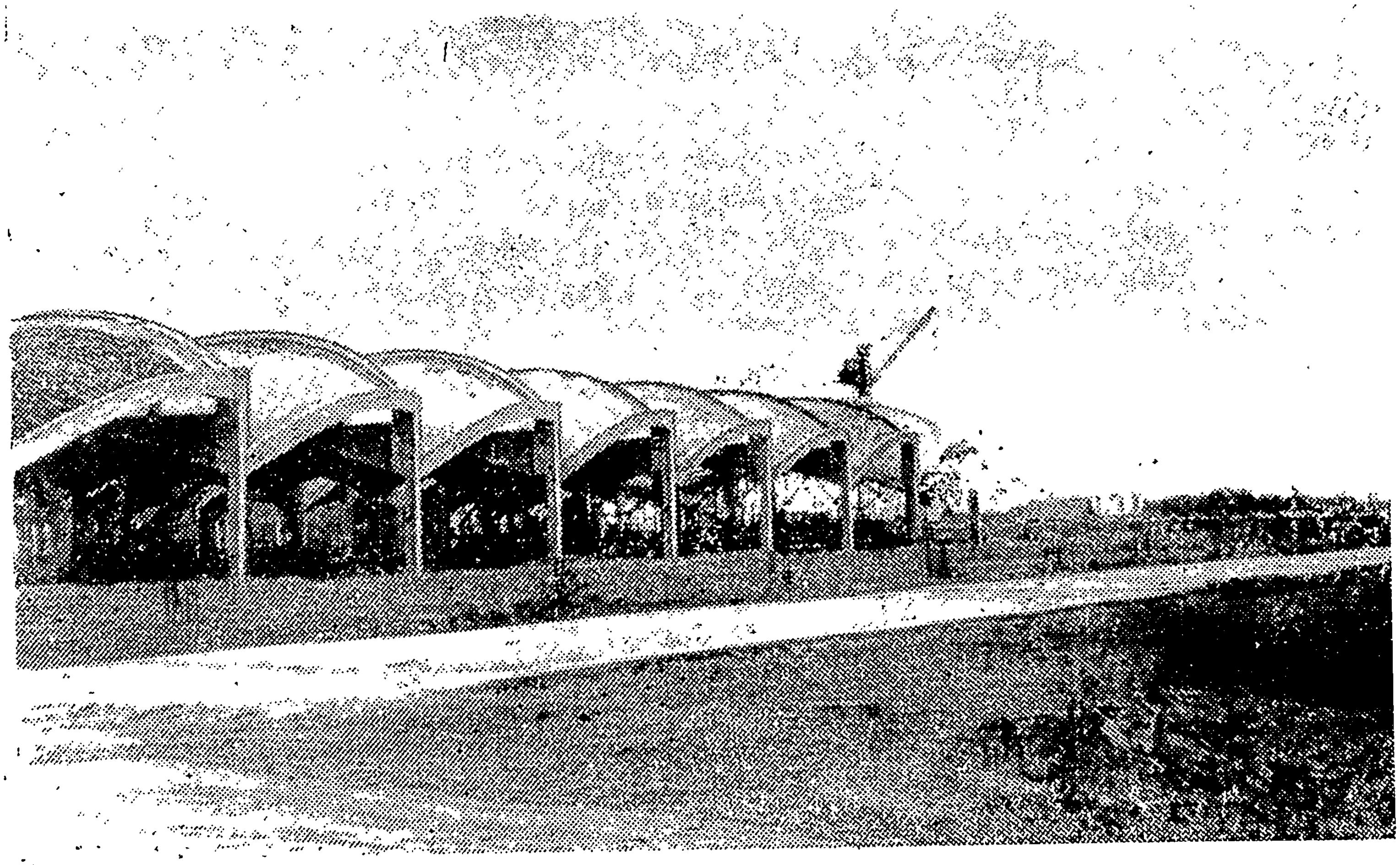
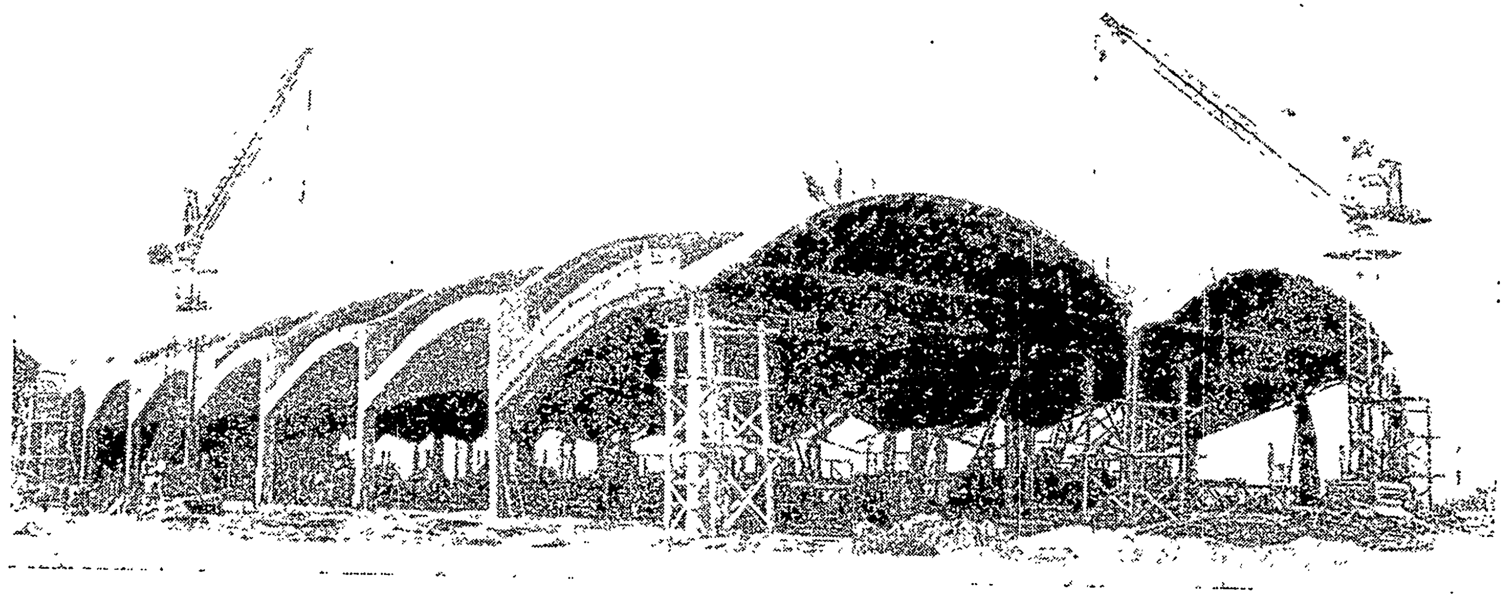


The plant under construction

DOUBLE CURVED ROOF TEXTILE PLANT AT MINSK
Source: Moscow TsINIS. Causes of Structural Failures, p. 53 (TH 3401 .M7)
PLATE 29



Declassified in Part - Sanitized Copy Approved for Release @ 50-Yr 2013/06/21 : CIA-RDP81-01043R002100010003-0

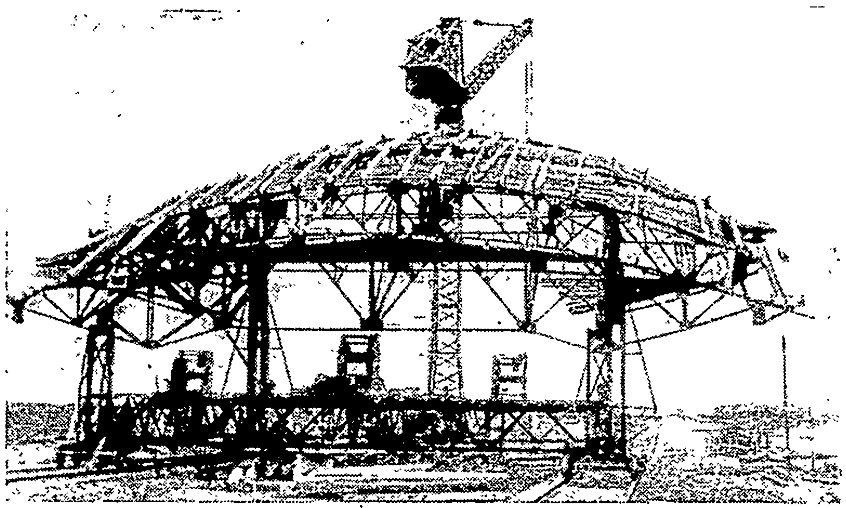


Movable concrete form for shell pouring in use

DOUBLE-CURVED ROOF PLANT IN THE PROCESS OF CONSTRUCTION, MINSK (?)
Source: Stroitel'naya Promyshlennost', No. 8, 1955, p. 10
PLATE 30



Declassified in Part - Sanitized Copy Approved for Release @ 50-Yr 2013/06/21 : CIA-RDP81-01043R002100010003-0



Frame of a movable concrete form for double curved roof shells

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Stroitel'naya Promyshlennost', No. 8, 1955, p. 10
PLATE 31

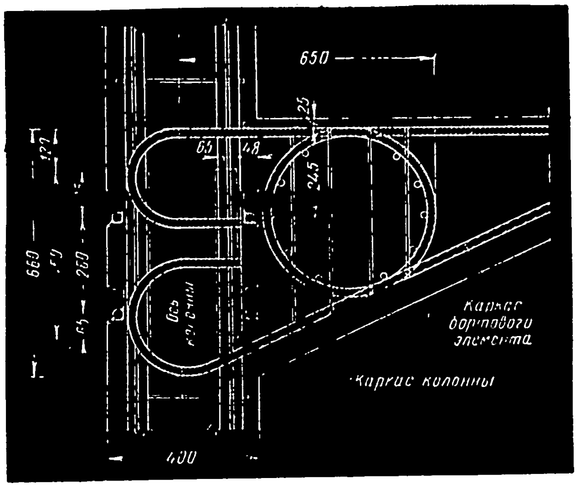



Fig. 1. Perederiy side member-column joint
1. Side member reinforcement
2. Column reinforcement

**Conversion Table**

| mm. | in. |
|---|---|
| 25 | 0.984 |
| 48 | 1.89 |
| 65 | 2.56 |
| 120 | 4.72 |
| 125 | 4.92 |
| 245 | 9.65 |
| 260 | 10.2 |
| 450 | 17.7 |
| 650 | 25.6 |
| 660 | 26.0 |

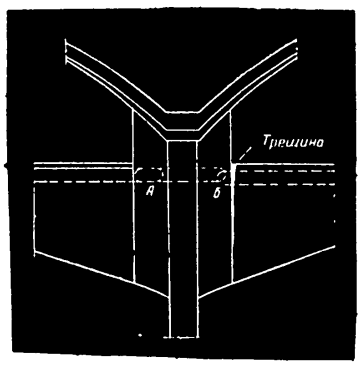



Fig. 2. Crack in a Perederiy tie beam-column joint
A. Normal position of reinforcement loops in the joint
B. Shifted loops in the joint

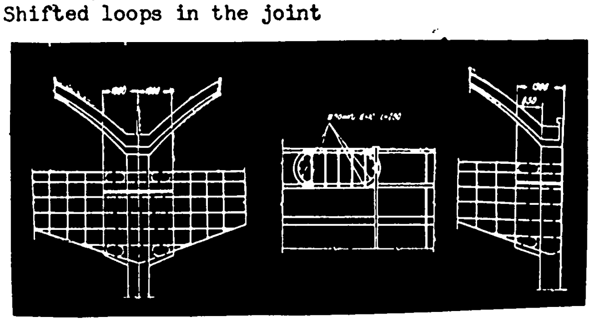


Fig. 3. Perederiy tie beam-column joint

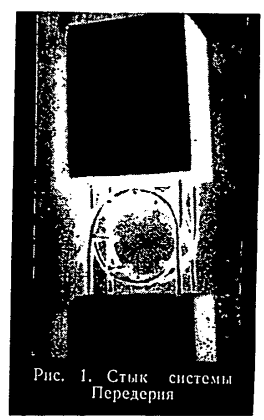



Fig. 4. Perederiy joint (photograph)

Perederiy Type Joint
DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK

Sources: Fig. 1. Moscow TsINIS. Causes of Structural Failures, p. 55. (TH 3401.M7)
Fig. 2-3 Stroitel'naya Promyshlennost', No. 2, 1957, p. 23
Fig. 4 Construction in Earthquake Affected Areas, p. 66.(NTOSP publication)

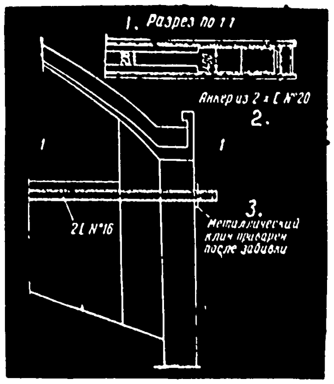



Fig. 1. Details of additional channel-steel tie beams
1. Section 1-1.
2. Steel channel No. 20, scrap pieces
3. Steel wedge welded to channels

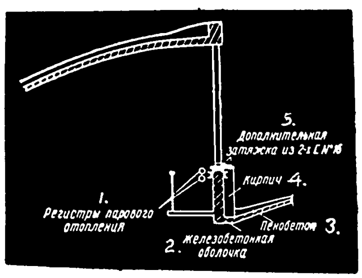



Fig. 2. Heating arrangement that speeded collapse
1. Pipe heaters
2. RC roof shell
3. Foam concrete
4. Brick
5. Additional tie beam (2 - No. 16 steel channels)

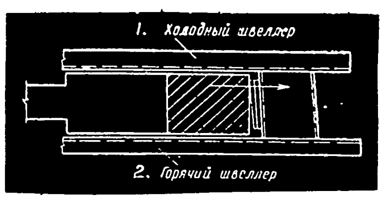



Fig. 3. Heater influence on additional steel tie beams
1. Cold steel channel
2. Heated steel channel

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Stroitel'naya Promyshlennost', No. 2, 1957, p. 23-24

PLATE 33



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General view of the collapsed part of the plant

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Moscow TsINIS. Causes of Structural Failures, p. 54. (TH 3401 .M7)
PLATE 34



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General view of the debris. The brick partition that stopped the chain collapse is seen in the background

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Stroitel'naya Promyshlennost', No. 2, 1957, p. 21
PLATE 35



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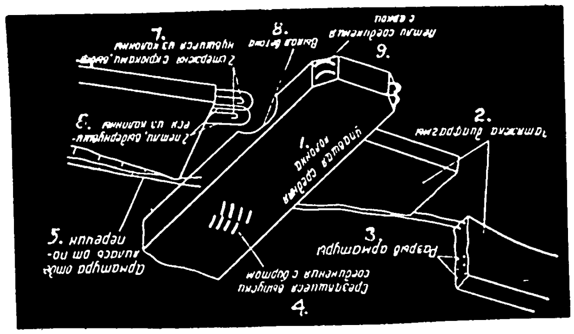

Fig. 1. Position of the column and tie beam in the bay where failure started
1. Fallen middle column. 2. Tie beam. 3. Ruptured reinforcement.
4. Remnants of sheared off reinforcement. 5. Reinforcement separated from stirrups. 6. Two loops torn out of the column. 7. Two bars torn out of the column. 8. Broken concrete. 9. Column-arch joint loops.
(Note that lettering on drawing is inverted)

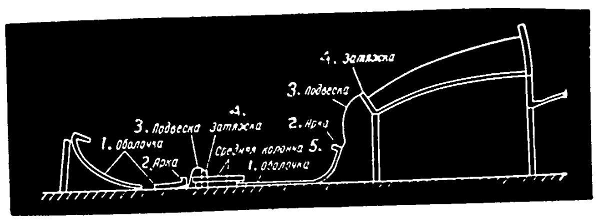


Fig. 2. Chain collapse. Relative position of various structural members
1. Roof shell. 2. Arch. 3. Suspension support. 4. Tie beam.
5. Middle column.

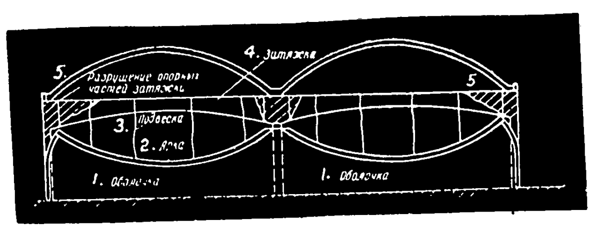


Fig. 3. Shell failure at the brick partition which stopped chain collapse
1. Shell. 2. Arch. 3. Hanger. 4. Tie beam. 5. Damaged tie beam ends

Development of the Failure

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK
Source: Stroitel'naya Promyshlennost', No. 2, 1957, pp. 24-25

PLATE 36

POOR ORIGINAL

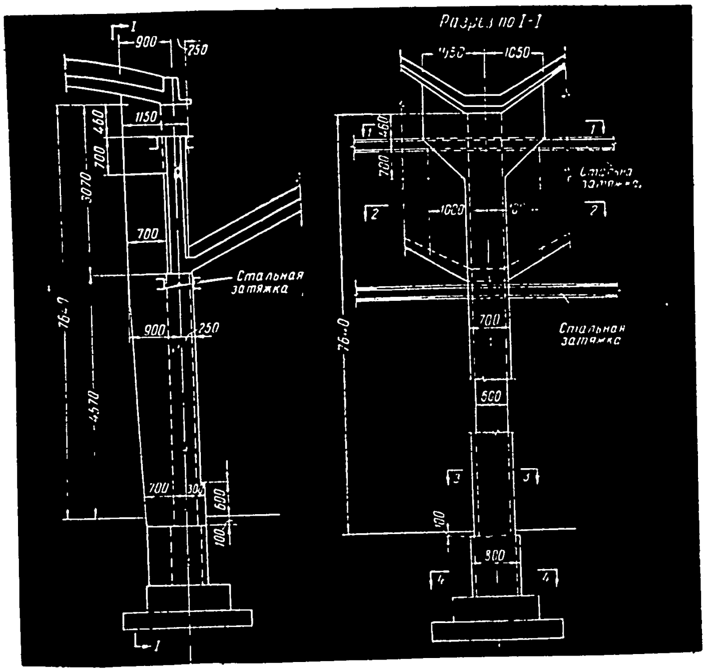


Fig. 1-2. Proposed strengthening: arch-tie beam elements-with steel channels; columns-with concrete casing; a) steel channels

**Conversion Table**

| mm. | in. |
|---|---|
| 100 | 3.94 |
| 200 | 7.87 |
| 250 | 9.84 |
| 300 | 11.8 |
| 350 | 13.8 |
| 400 | 15.7 |
| 460 | 18.1 |
| 500 | 19.7 |
| 600 | 23.6 |
| 700 | 27.6 |
| 800 | 31.5 |
| 900 | 35.4 |
| 950 | 37.4 |
| 1,000 | 39.4 |

| mm. | ft. |
|---|---|
| 1,050 | 3.45 |
| 1,150 | 3.78 |
| 2,000 | 6.56 |
| 2,100 | 6.89 |
| 3,070 | 10.1 |
| 4,570 | 15.0 |
| 7,640 | 25.1 |

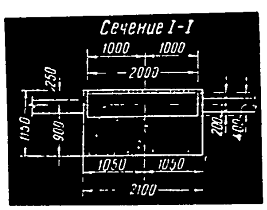


Fig. 3. Section 1-1

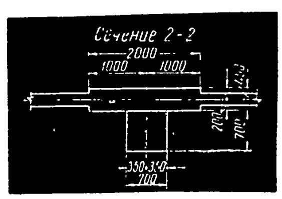


Fig. 4. Section 2-2

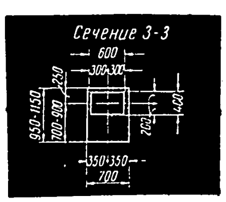


Fig. 5. Section 3-3

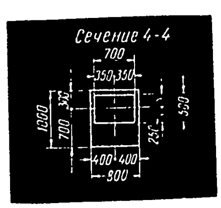


Fig. 6. Section 4-4

Figs. 3-6. Strengthened columns

Proposed Strengthening of Arch-tie Beam Elements and Columns

DOUBLE-CURVED ROOF TEXTILE PLANT AT MINSK

Source: Moscow TsINIS. Causes of Strcutrual Failures, pp. 57-58. (TH 3401.M7)

# CHAPTER X

## COLUMN DETERIORATION IN A REINFORCED CONCRETE HYPERBOLIC COOLING TOWER

### Foreword

The following account is based on a source of very limited scope. It provides the description of columns and column defects in a hyperbolic cooling tower but gives no description of the tower as a whole.

To put these columns in their proper perspective, photographs and structural description of an identical (or a very similar) hyperbolic cooling tower are provided from another source.

For this reason, the account is divided in two parts.

## A. DETERIORATION OF REINFORCED CONCRETE COLUMNS IN A HYPERBOLIC COOLING TOWER.

### Location

Heat and Electric Power Plant (TETs) No. 16, Mosenergo.

### Structure

Inclined precast reinforced concrete columns (72 in number) supporting the shell of a reinforced concrete hyperbolic cooling tower.

### Construction

Columns. The precast reinforced concrete columns are octagonal in section, 340 mm (13.4 in.) across (Plate 41, fig. 2).

Column reinforcement. The reinforcement consists of:

a) 8 deformed bars, diameter 24 mm (0.945 in.);
b) wire, diameter 8 mm (0.315 in.), spiral pitch 100 mm (3.94 in.).

Column concrete as specified. The designers specified Mark 140 concrete on puzzuolan-based portland cement.

Column precasting as actually carried out. The columns were precast of Mark 200 concrete in the yard.

As the designers had failed to indicate the frost-resisting requirements for the concrete, the builders, presumably at the yard, did not take this factor into account. The choice of the composition of concrete was made on the basis of Mark 400 puzzuolanic portland cement.

Data on Concrete of the Columns.

| Material | Data |
|---|---|
| Cement | Mark 400 puzzuolanic portland cement<br>Cement test cone slump 4-6 cm. (1.58 - 2.36 in.)<br>Water - cement ratio 0.59 (6.65 gal/sack) |
| Sand | Modulus of coarseness 2.6<br>Particles smaller than 0.15 mm. (0.006 in.) 4%<br>Organic content ------ within allowable limits<br>(In spite of laboratory instructions the sand was not sifted before use) |
| Gravel | Quite dirty. Sand content - 20%<br>Impurities (clay, silt, etc.) - 11.3%<br>(Laboratory instructions to wash the gravel before use were unheeded) |
| Concrete | Nominal composition by volume: 1:1.54:3.04<br>Materials kg/m³<br>cement 300 kg. (506 lb/yd³)<br>sand 601.8 kg. (1,020 lb/yd³)<br>gravel 1,208 kg. (2,040 lb/yd³)<br>Ultimate strength of test specimens:<br>1 week - 113.4 kg/cm² (1,610 lb/in²)<br>4 weeks - 198.6 kg/cm² (2,820 lb/in²) |
| Concrete preparation | In concrete mixer; cement dosage by weight. |
| Concrete tamping | With I-21 vibrators. |
| Concrete curing | In steam chamber during 18-24 hrs at 70° - 80°C (158 - 176°F). Raising of temperature to the desired degree took 2 hours. The strength of concrete at the time of release for delivery fluctuated from 80 to 190 kg/cm² (1,140 to 2,700 lb/in²). |

Most of the columns were made of the above described concrete; a few of the presumably "commercial" concrete based on portland cement.

Construction time table. The columns were precast in February and March, 1954. The cooling tower became operative in June 1955.

## Deterioration of Columns

In the spring of 1957, extensive deterioration of the surface of the columns was noted; particularly of the columns exposed to the down-flow of the cooled water.

The surface was covered with cracks. Rather thick pieces of concrete, some firm, some crumbling, could be easily separated from the surface. Spiral reinforcement was bare in many columns (Plate 38, fig. 1). The cracked concrete in lower parts of the columns was covered in places with porous, yellow-gray deposits; apparently, this was calcium carbonate formed as a result of leaching of concrete by water penetrating into the cracks (Plate 38, fig. 2). Some columns without perceptible cracks had damaged surfaces to depths of 5-7 cm.(2-2.8 in.) thus exposing not only the spiral but also the bar reinforcement.

## Causes of the Column Deterioration

The deterioration of columns was mainly caused by the alternate freezing and thawing of the porous concrete saturated with water.

Leaching had only a secondary effect, since it affected the already cracked concrete.

The wetting of columns could have been avoided had the water-deflecting arrangements been built strictly according to specifications.

The low frost-resisting quality of concrete was due to:

a) inadequate density of the structure of concrete resulting from the use of puzzuolanic cement.

b) too high a water - cement ratio.

c) dirty gravel

d) too fast a rate of cooling of columns upon their removal from the steam chamber.

## Repair of the Columns

The affected columns were cleaned, notched and washed. The less deteriorated columns were gunited over mesh of the "Rabits" type and iron-plated; the more damaged columns were concreted in concrete forms.

(Note: Two similar cooling towers, one of which went into service in 1952 and the other in 1955, had columns poured in place. So far (the source of information was published in September 1957), those columns are in satisfactory condition).

## B. STRUCTURAL DETAILS OF A REINFORCED CONCRETE HYPERBOLIC COOLING TOWER

### Location

Unidentified TETs in the Middle Zone of the European USSR.

### Structure

Natural draft reinforced concrete hyperbolic cooling tower of a drop-and-film type.

### Construction

Basically, the structure is a somewhat modified conventional wooden cooling tower enveloped by a shell in the form of a hollow hyperboloid of revolution which provides natural draft.

Description of its construction may therefore, be divided in 2 parts:

I. Structural details of the hyperboloid shell.
II. Construction of the conventional part.

Vertical and cross sections of the tower are shown on Plate 39.

A photograph of the tower in operation appears on Plate 40.

### I. Structural Details of the Hyperboloid Shell

General statistical data:

| | |
|---|---|
| Height of the superstructure | 55.3 m (181 ft.) |
| Height of the substructure | 2.25 m (7.4 ft.) |
| Shell diameter (outline) | 50.4 m (165 ft.) |
| Area occupied by the cooling tower | 1,994 $m^2$ (21,500 $ft^2$) |
| Maximum capacity | 12,000 $m^3$/hr (53,000 gpm) |

Foundation. The lower abutment ring or the foundation ring of the shell is of Mark 140 reinforced concrete. It contains 375 $m^3$ (490 $yd^3$) of reinforced concrete. It has a T-shaped profile and rests on a "concrete preparation" (apparently stabilized earth) with toothed profile which required 400 $m^3$ (520 $yd^3$) of concrete. Maximum pressure on the soil at this point is 0.8 kg/$cm^2$ (11.5 lb/$in^2$) under windy conditions. Besides providing the foundation for the tower, the lower abutment ring forms the wall of the tower's water basin. The ring is provided with a lip with expansion joints. These joints are made during the pouring of concrete, when pieces of plywood wrapped in tar paper are introduced at proper intervals; after 5-7 days they are removed and the space thus formed is filled with bitumen.

Vertical section of the abutment ring is shown on Plate 41, fig. 1.

As the "concrete preparation" and the ring were built under winter conditions a light heating arrangement had to be made around it. The heater consisted of plywood sheathing fastened to the concrete form, and steam coils.

A photograph of the arrangement is shown on Plate 41, fig. 3.

Columns. Seventy-two inclined octagonal columns are of Mark 170 reinforced concrete, requiring 27 m3(35 yd3) of concrete. Column reinforcement is described on page of this report. The columns rest on the lower abutment ring (Plate 41, fig. 1), They were poured simultaneously with the upper abutment ring; the pouring was continuous.

Upper abutment ring. The upper abutment ring consists of 40 $m^3$ (52 $yd^3$) of Mark 170 reinforced concrete. The ring ties the tops of the columns and forms the base of the hyperbolic shell; transfers the load of the shell to the substructure.

Hyperboloid shell. The shell is of reinforced concrete containing 767 $m^3$ (1,000 yds3).

Cement composition in the preparation of concrete was the following:

| | |
|---|---|
| Portland cement | 66.7% |
| Puzzuolanic portland cement | 33.3% |
| Tripolite | 10.0% |
| Cement obtained | Mark 350 |

The size of the gravel varied from 20 to 40 mm. (0.8 - 1.6 in.) depending on the thickness of the part of the shell for which it was prepared. The adopted water-cement ratio was presumably 0.5.* Slump of standard concrete test cone was 7-8 cm. (2.8-3.2 in.).

The thickness of the shell at the base is 350 mm. (13.8 in.), it gradually decreases upwards to 100 mm.(3.94 in.) at the height of 20.3 m. (66.6 ft.); from that point, the thickness is constant to the rim where there is a stiffening ring.

A catwalk with steel railing is built on the rim.

Lightning protection of the shell. The catwalk railing is connected with the reinforcement of the inclined columns by means of 6 lap-welded rods within the shell wall.

Shell coating. The coating for the internal surface of the shell has the following components:

| | |
|---|---|
| No. 3 Bitumen | 30% |
| No. 5 Bitumen | 20% |
| Kerosene or benzine (gasoline) | 50% |

*5.6 gal/sack

The two kinds of bitumen are heated to 170-180°C (338-356°F), thoroughly mixed, cooled to 120°C (248°F) and then diluted with kerosene or gasoline. The mixture is applied to the surface twice at the temperature of 70°C (158°F) with a compressed-air sprayer.

The outside surface is smoothed and sprayed with cement.

The cooling tower basin. As previously indicated, the circular wall of the basin is formed by the lower abutment ring, i. e. is of Mark 140 reinforced concrete. The bottom of the basin is presumably also of Mark 140 reinforced concrete; it was apparently poured in two lifts and covered with moisture-resistant material. It provides support for precast Mark 140 reinforced concrete posts which carry the wooden frames of the conventional part of the tower. The basin is divided in two sections by a reinforced concrete partition. The depth of each section is 1.8 m. (6 ft.). The useful capacity of the basin is 1,600 $m^3$ (423,000 gal.).

The construction of the basin bottom, basin partition and the posts supporting the wooden frames required 543 $m^3$(709 $yd^3$) of reinforced concrete.

Water inflow and discharge channels. Water flows from the TETs to the cooling tower through two 1.9 x 0.88 m. (6.2 x 2.9 ft.) Mark 140 reinforced concrete channels. One meter (3.28 ft.) steel pipes with their stuffing boxes imbedded in the walls of the channels are interposed between the ends of the channels and the tower. Just outside the tower the pipes are provided with sliding baffles. Experience showed that a warming arrangement should be made around the baffles for winter conditions.

The two sump discharge channels are also of Mark 140 reinforced concrete; they are presumably of the same size as the water inflow channels.

## II. Construction of the Conventional Part of the Cooling Tower.

A photograph of a sectional model of this part appears on Plate 42. It shows that this modified conventional part consists of a centrally located water tower which is surmounted by a water-distributing reservoir and surrounded by wood filling where the cooling of water actually takes place.

The water tower is of Mark 140 reinforced concrete containing 51 $m^3$(67 $yd^3$) of reinforced concrete, water inflow channels included. The tower is divided in 2 parts by a vertical reinforced concrete partition; this permits half the tower to be put out of action when necessary. The water distributing reservoir on the top of the tower has 18 sides with openings. Water passes through these openings over the wooden baffles into the water distributing troughs, from which it spreads over and trickles through the filling.

The wood filling represents a structure consisting of boards, triangular rods, distributing troughs and splash troughs with plastic bungs, the whole being supported by frames which rest on precast reinforced concrete posts anchored at the bottom of the tower water basin.

The structure may be regarded as composed of 3 zones.

Zone 1 extends around the periphery; it consists of panels made of boards 10 mm. (0.394 in.) thick. The panels are distributed over 3 levels at an angle of 61° with the horizontal. Zone 3 is next to the water-distributing tower. Its boards are placed tangentially. They are 10 mm. (0.394 in.) thick, form a 61° angle with the horizontal and are presumably distributed over 3 levels. Zone 2 is located under zones 1 and 3; it consists of triangular tangentially placed rods at 8 levels.

The filling is of pine; all its joints are made with galvanized iron bolts and nails; it occupies an area of 1520 $m^2$ (16,400 $ft^2$) and requires 827 $m^3$ (350,000 bd. ft.) of lumber for its erection.

## Construction Time Table

Apparently, this is the first hyperbolic cooling tower ever to be built in the Soviet Union. It was erected according to the following schedule:

| | |
|---|---|
| Earthwork | Nov. - Dec. 1948 |
| Lower abutment ring | Jan. - Feb. 1949 |
| Upper abutment ring and the columns | Apr. - May 1949 |
| Hyperbolic shell | May - Aug. 1949 |
| Wood filling section | Sep. 1949 - Jan. 1950 |

In March 1950, the tower was operating at its full capacity.

## Note

A cooling tower with a hyperbolic shell of factory-manufactured precast reinforced concrete elements was erected in Hungary before 1956. The method of its construction is under study with a view to introducing it in the Soviet Union.

## Source

1) Beton i Zhelezobeton, No. 9, 1957, pp. 368-369
2) Otlivnoy, I. F., Building Reinforced Concrete Hyperbolic Cooling Towers, pp. 1-48. TJ 563.08
3) Stroitel'naya Promyshlennost', No. 8, Aug. 1955, p. 22

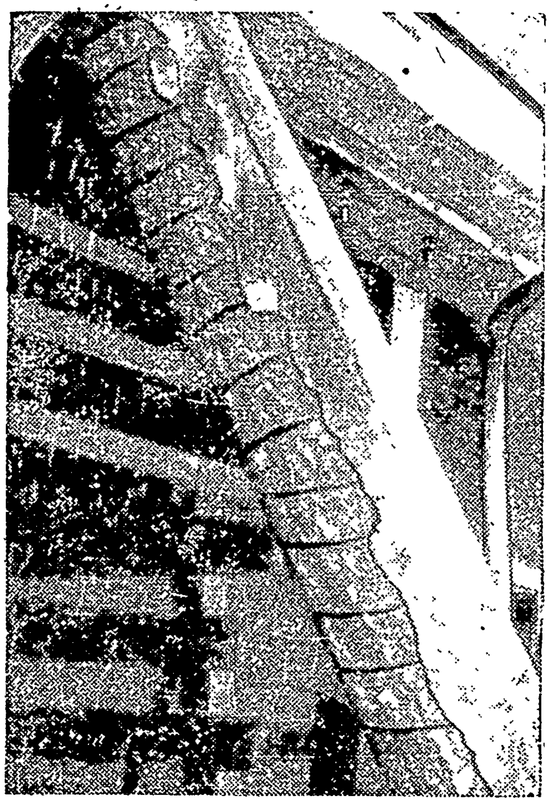

Fig. 1. Effects of frost

Fig. 2. Crumbling concrete; calcium carbonate deposits

DETERIORATION OF REINFORCED CONCRETE COLUMNS IN A HYPERBOLIC COOLING TOWER
(Heat and Electric Power Plant No. 16, Mosenergo)
Source: Beton i Zhelezobeton, No. 9, 1957, p. 369
PLATE 38



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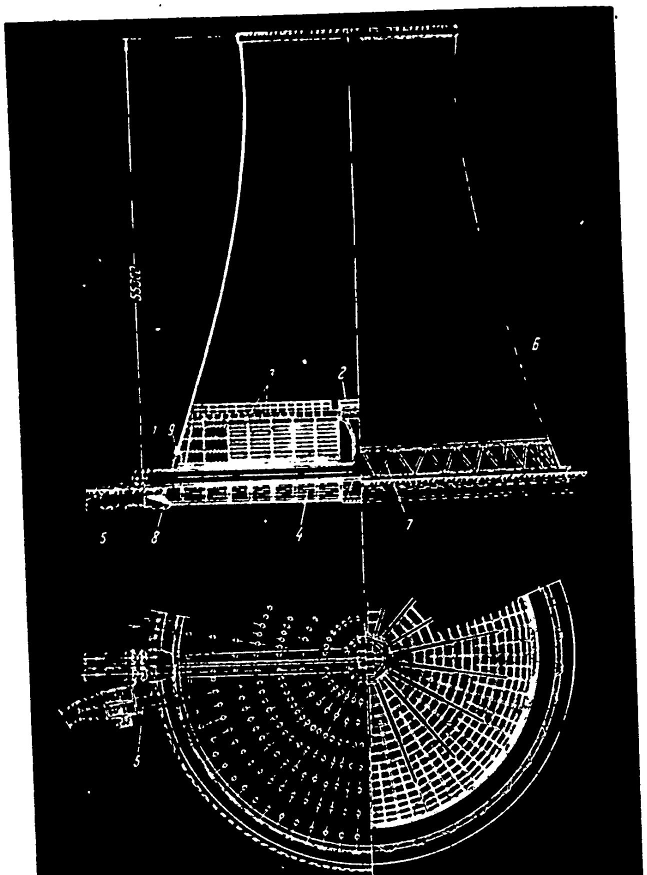


Фиг. 2.

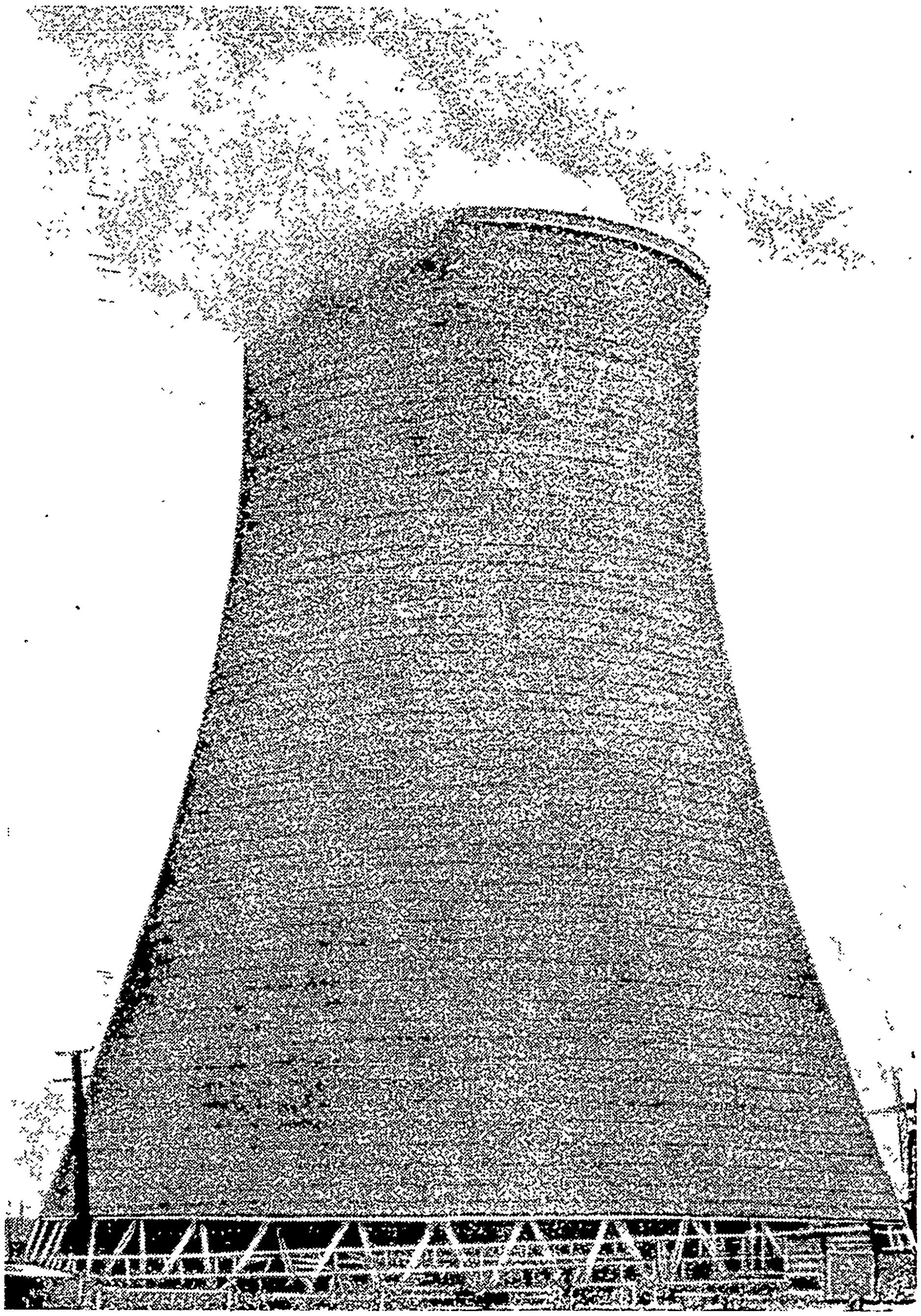
1. Water inflow channel. 2. Water distributing tower. 3. Wood filling.
4. Tower basin. 5. Water discharge channel. 6. Reinforced concrete hyperbolic shell. 7. Concrete columns. 8. Lower abutment ring.
9. Upper abutment ring.

Vertical and Cross Sections

REINFORCED CONCRETE HYPERBOLIC COOLING TOWER

Source: Otlivnoy, I. F., Building Reinforced Concrete Hyperbolic Cooling Towers, p. 5, (TJ 563.08)

PLATE 39



The Tower in Operation

REINFORCED CONCRETE HYPERBOLIC COOLING TOWER

Source: Otlivnoy, I.F., Building Reinforced Concrete Hyperbolic Cooling Towers, p. 4, (TJ 563.08)

PLATE 40



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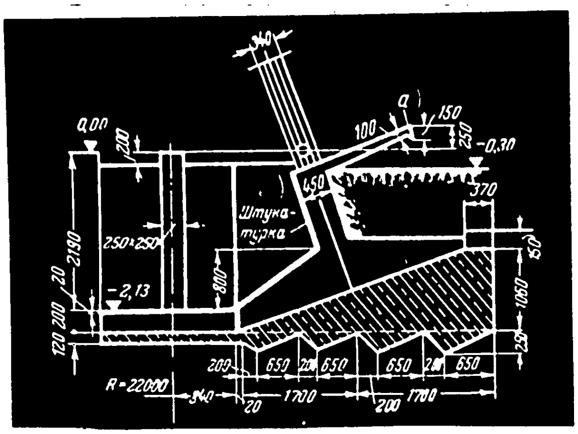



Fig. 1. Lower abutment ring; vertical section.
a) Abutment ring lip. b) Parging.

Conversion Table

| mm. | in. |
|---|---|
| 20 | 0.787 |
| 100 | 3.94 |
| 120 | 4.72 |
| 140 | 5.51 |
| 150 | 5.91 |
| 200 | 7.87 |
| 250 | 9.84 |
| 340 | 13.4 |
| 370 | 14.6 |
| 450 | 17.7 |
| 650 | 25.6 |
| 840 | 33.1 |
| 1060 | 41.7 |
| 1700 | 67.0 |
| 2190 | 86.3 |

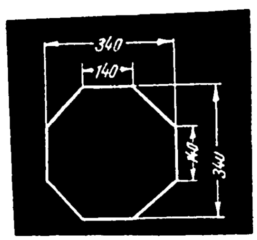



Fig. 2. Column cross section.

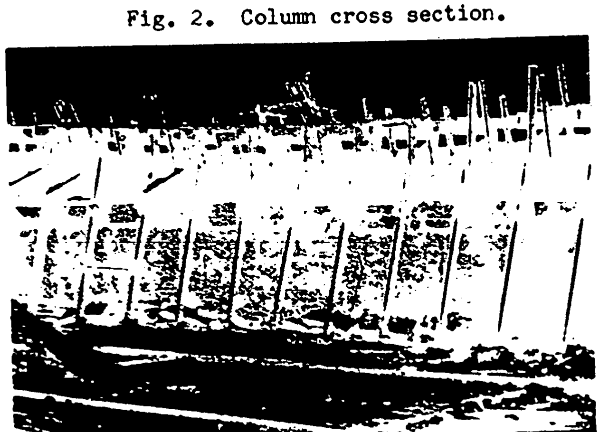
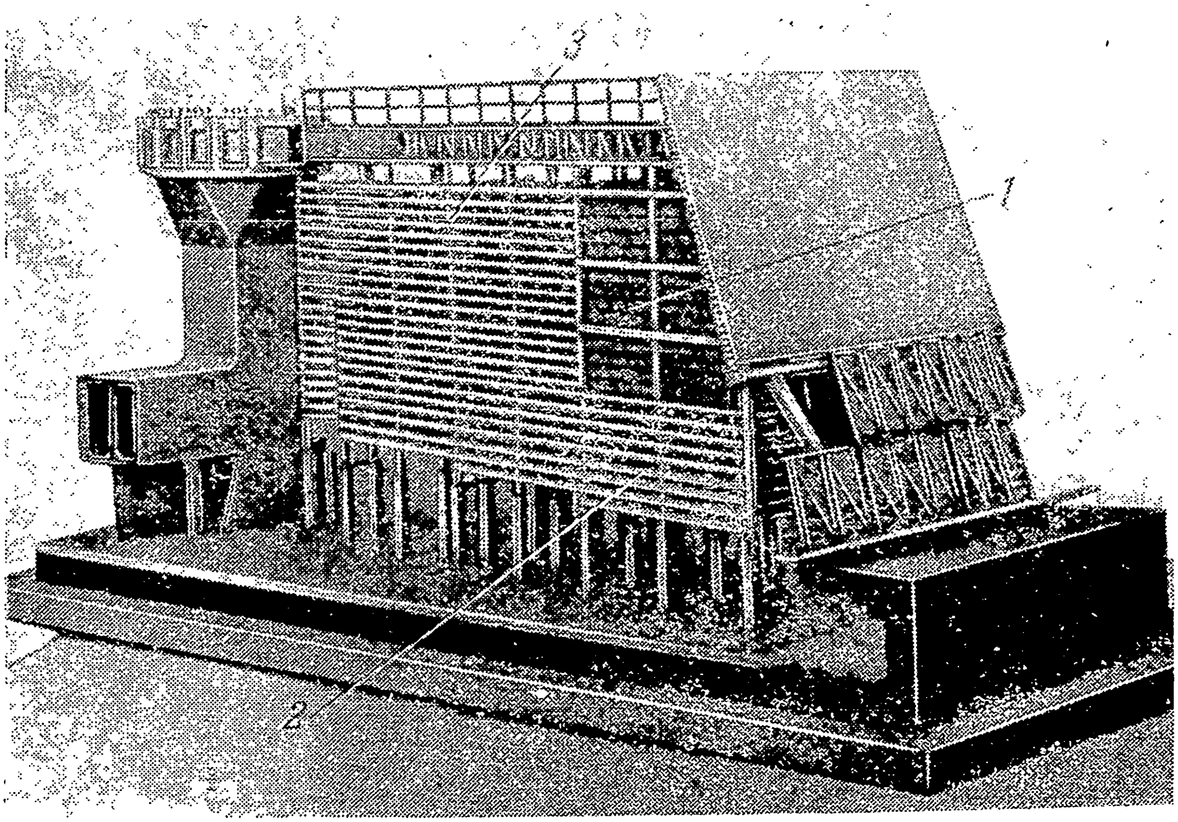


Fig. 3. Heating arrangement for the pouring ring under winter conditions

REINFORCED CONCRETE HYPERBOLIC COOLING TOWER
Source: Otlivnoy, I. F., Building Reinforced Concrete Hyperbolic Cooling Towers, pp. 6 &14, (TJ 563.08)

PLATE 41




Modified conventional part of the tower; sectional model.
(For 1, 2, and 3 - Filling zones - See text, p. 73).

REINFORCED CONCRETE HYPERBOLIC COOLING TOWER
Source: Otlivnoy, I.F., Building Reinforced Concrete Hyperbolic Cooling Towers, p. 8, (TJ 563.08)

PLATE 42

# CHAPTER XI

## FAILURE OF A PROP STORAGE WING (MOSCOW)

### Location

"Mosfilm" cinema studio lot, Potylikha, Moscow.

### Structure

One-story three-aisle brick structure with load-bearing walls and two rows of load-bearing interior piers.

### Construction

Plan and transverse section of the structure are shown on Plate 43 and Plate 44 respectively.

Dimensions. Dimensions of the structure are:

| | |
|---|---|
| Length (according to text) | 22.8 m. (74.7 ft.) (according to plan it is 69.0 ft.) |
| Width | 22.1 m. (72.5 ft.) |
| Total height (middle aisle) | 9.05 m.(29.7 ft.) |
| Total height (side aisles) | 5.07 m.(16.6 ft.) |
| Aisle width | 7.36 m.(24.2 ft.) |
| Bay length | 6.0 m.(19.7 ft.) |

Foundations. Foundations under the walls are of rubble concrete; foundations under the interior piers are of monolithic reinforced concrete.

Walls. The walls are of Mark 100 brick laid in Mark 50 mortar. They are 51 cm. (20.1 in.) thick. There is a clearstory above the side aisles.

Interior piers. The piers of 64 x 64 cm. (25.2 x 25.2 in.) cross section are of Mark 100 brick laid in Mark 50 mortar; they are reinforced with steel mesh.

Beams. Longitudinal beams which provide support for the clearstory are of monolithic reinforced concrete. They rest on reinforced concrete bearing pads surmounting the piers.

Roof. Roof beams are of precast reinforced concrete. They support precast reinforced concrete roof slabs.

Roofing. Presumably ruberoid; heat insulation - slag.

Construction time table. The piers and the walls of side aisles were erected during the spring and summer of 1955. The clearstory and the roof slabs were laid in the winter of 1956.

## Failure of the Structure

On 11 April 1956, the structure failed. First, three interior piers broke in the vicinity of beam-pier joints (intersection of axes B, C, D, with axis 9, Plate 43); then followed the collapse of the longitudinal beams, wall, roof beams and some 300 $m^2$(3,230 $ft.^2$) of roof slabs.

The collapsed structure is shown on Plate 45; a detail appears on Plate 46.

## Causes of Collapse

On the day of collapse, there was a steel scaffolding and a quantity of building material stored on the roof between axes 8 and 9 (Plate 44). This undoubtedly increased the load borne by the piers along axis 9, but the main causes of failure seem to be the following:

1. Inadequate quality of brick. Design required that brick both solid and hollow be of Mark 100. The brick supplied by two factories was accompanied by certificates to the effect that its quality was Mark 100. Post-failure examination, however, disclosed that solid brick was of Mark 50, the hollow of Mark 75. This by itself was sufficient to lower the working strength of the brickwork laid in Mark 50 mortar by some 20-35%.

For Mark 50 mortar the Soviet "Norms and Technical Conditions", NiTU-120-55, established the following relationship between 3 kinds of brick and brickwork working strength :

| Mortar Mark | Brick Mark | Working Strength |
|---|---|---|
| 50 | 100 | 30 $kg/cm^2$ - 426 $lb/in^2$ |
| 50 | 75 | 25 $kg/cm^2$ - 355 $lb/in^2$ |
| 50 | 50 | 20 $kg/cm^2$ - 284 $lb/in^2$ |

2. Failure to reinforce the pier brickwork. Although the design called for a 5 x 5 cm. (1.97 x 1.97 in.) steel mesh 6 mm. (0.236 in.) thick reinforcement of bed joints spaced at 15 cm. (5.91 in.) along the height of the piers, no reinforcement at all was found in the piers.

3. Faulty execution of pier bed joints. They were found to be dry to a depth of 2 cm. (0.787 in.). With pier cross section area of 4,096 $cm^2$ (634.9 $in^2$) and dry joint area of some 512 $cm^2$(79.4 $in^2$) compressive stresses in the brickwork rose correspondingly.

4. Eccentric load on two foundations at their pier-foundation joints. The piers at the intersection of axes B and D with axis 9 (Plate 43) were shifted some 16-17 cm. (6.3-6.69 in.) in opposite directions with respect to foundation axis 9. This was the source of additional stresses in the piers.



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## Reconstruction

The crushed brick piers were replaced with 40 x 40 cm. (15.8 x 15.8 in.) reinforced concrete columns.

The brick piers remaining standing were strengthened at four corners with 75 x 8 mm. (2.95 x 2.95 x 0.315 in.) steel angles held in place by welded horizontal steel bands.

The roof was reconstructed according to the original design.

## Note

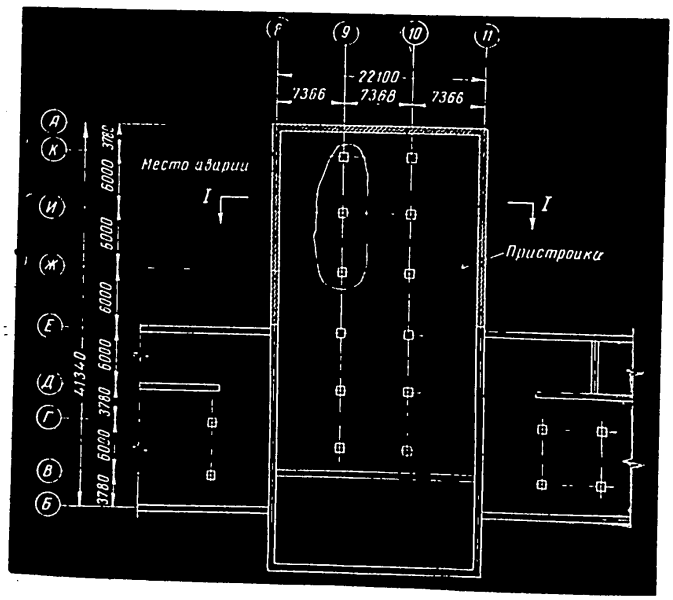
The factories delivered the wrong quality brick under cover of correct certificates.

The builders, for their part, did not check the mechanical qualities of the brick; disregarded the specifications with respect to reinforcement of the piers; left pier bed joints partially dry and constructed at least 2 faulty pier-foundation joints.

Assuming that the design was adequate, negligence on the part of the brick factory and builders is at the bottom of the accident.

## Source

Moscow TsINIS. Causes of Structural Failures, pp. 25-29.
TH 3401.M7




a) The new wing. b) Piers that failed

Plan of the Structure

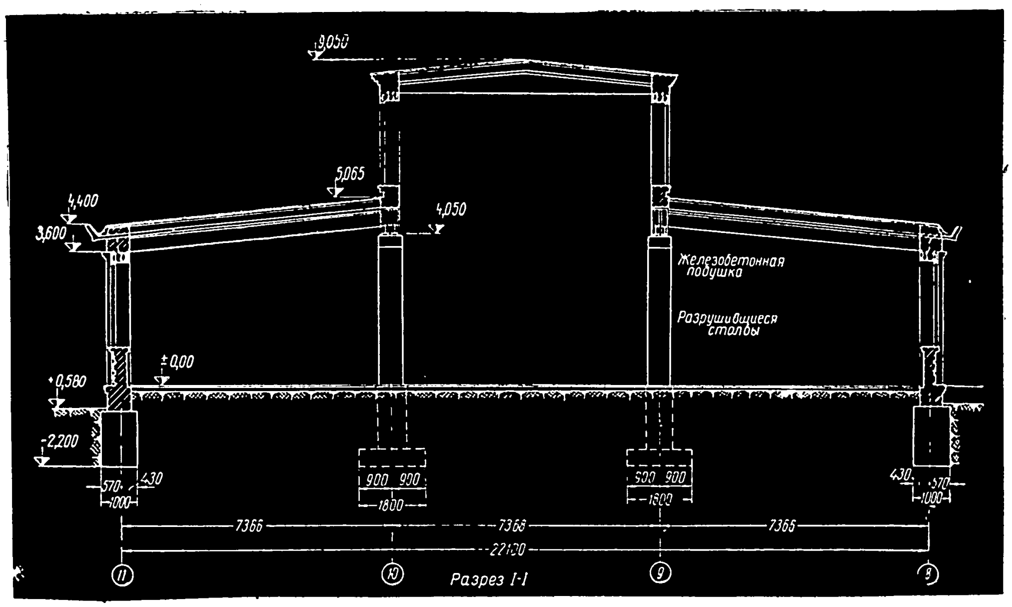
Conversion Table

| mm. | ft. |
|---|---|
| 3,780 | 12.4 |
| 6,000 | 19.7 |
| 7,366 | 24.2 |
| 22,100 | 72.5 |
| 41,340 | 136 |

PROP STORAGE WING, "MOSFILM" STUDIO, MOSCOW

Source: Moscow TsINIS. Causes of Structural Failures, p. 25. (TH 3401.M7)

PLATE 43




1. RC bearing pad. 2. Collapsed piers
Section 1-1

Conversion Table

| mm. | ft. |
|---|---|
| 430 | 1.41 |
| 570 | 1.87 |
| 900 | 2.95 |
| 1,000 | 3.28 |
| 2,200 | 7.22 |
| 3,600 | 11.8 |
| 4,050 | 13.3 |
| 4,400 | 14.4 |
| 5,065 | 16.6 |
| 7,366 | 24.2 |
| 9,050 | 29.7 |
| 22,100 | 72.5 |

PROP STORAGE WING, "MOSFILM" STUDIO, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 26. (TH 3401.M7)

PLATE 44

General view after the collapse

PROP STORAGE WING "MOSFILM" STUDIO, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 28.(TH 3401.M7)

PLATE 45



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Failure of a beam-pier joint

PROP STORAGE WING "MOSFILM" STUDIO, MOSCOW
Source: Moscow TsINIS. Causes of Structural Failures, p. 29, (TH 3401.M7)

PLATE 46

## CHAPTER XII

## FAILURE OF A BRICK WALL PIER IN A HAND GAME TRAINING HALL (MOSCOW)

### Location

Hand game stadium in Moscow.

### Structure

One-story lean-to of mixed construction presumably adjoining one of the end structures of the stadium. It accommodates two identical training halls.

### Training Halls

The halls are symmetrically located with respect to the longitudinal axis of the stadium. They are separated from each other by two brick walls; an automobile road passes between these walls to the stadium. Each hall has the following dimensions:

| | |
|---|---|
| Width | 16 m. (52.5 ft.) |
| Length | 28 m. (91.9 ft.) |
| Height to ceiling | 6.5 m. (21.3 ft.) |

Plan and transverse section of the damaged hall are shown on Plates 47 and 48 respectively.

Walls. Blank end walls are of brick; they are 51 cm. (20.1 in.) thick and are faced with L-shaped ceramic tiles 6 cm. (2.36 in.) thick.

The longitudinal wall on the stadium side represents a precast reinforced concrete frame with brick filling.

The outside longitudinal wall is 64 cm. (25.2 in.) thick; it is built of brick and faced with L-shaped ceramic tile 6 cm. (2.36 in.) thick; it has 64 x 103 cm. (25.2 x 40.6 in.) wall piers which are also built of brick, but with "zigzag" steel mesh reinforcement.

Roof. The roof is composed of the following elements:

1) Light steel trusses with parallel chords, span - 16 m. (52.5 ft.), height - 2.42 m. (7.95 ft.). Bay length - 6 m. (19.7 ft.). One end of the trusses rests on the reinforced concrete columns, the other on wall piers. As originally planned, the truss load was to be transferred to the piers through 50 x 50 cm. (19.7 x 19.7 in.) reinforced concrete bearing pads, but in the course of the construction, 44 x 65 cm. (17.3 x 25.6 in.) steel bearing plates 5 cm. (1.97 in.) thick were substituted for the pads.

2) Channel steel purlins resting on the upper truss chords and providing support for the roof cover.

3) Roof cover which consists of precast reinforced concrete slabs.

4) Roofing presumably of ruberoid.

Ceiling. The ceiling is of the suspended type. Precast reinforced concrete panels 198 x 39.5 cm. (78 x 15.5 in.) and 8 cm. (3.15 in.) thick are laid on welded supports consisting of a channel steel beam No. 30 a (weight - 23.2 lbs per ft.; channel depth - 11.7 in.; flange width - 3.32 in.; web thickness - 0.294 in.; area-6.8 in$^2$; See: SES Report No. 1, Table 1.0253B) and a 65 x 65x6 (2.56 x 2.56 x 0.236 in.) angle which are bolted to the gussets of the lower truss chords. The ceiling panels are covered with a moisture-resisting layer, slag wool heat insulation panels 10 cm. (3.94 in.) thick, and a 3 cm. (1.18 in.) thick layer of cement.

Construction time table. By the end of February 1956, the following elements of the structure were erected: walls, trusses, and roof cover slabs; the fully insulated ceiling was suspended from the lower truss chords. It was planned to complete the wall brickwork before the arrival of cold weather, sometime in October 1955, but actually the brickwork was laid by freezing method in November-December at temperatures below 32°F.

## Failure of the Pier.

On 1 March 1956, the pier at the intersection of axes P - 22 (Plate 47) collapsed. As a result, the truss supported by the pier also collapsed and so did two bay lengths of ceiling and roof slabs.

## Causes of Failure

Post-failure investigation revealed the following facts as regards materials and peculiarities of the construction work:

### 1. Mortar

a) No observations of the mortar temperature were made during the brick laying, nor were mortar specimens prepared or tested.

b) According to the work progress log, the composition of the mortar was cement-sand in the ratio of 1:5. On the basis of weight and volume of the mortar components, the laboratory did establish that the log record was correct. The exact quality of cement, however, could not be established. It was presumed that cement was the slowly hardening puzzuolanic portland cement, Mark 400 which just happened to be delivered to the site at the time. The mechanical properties tests of the mortar gave the following results:

| Specimen | Kind of Specimen | Ultimate compressive strength | |
|---|---|---|---|
| | | $kg/cm^2$ | $lb/in^2$ |
| 1 | Hand molded cubes cured in moist chamber for 3 days. | 3.7 | 52.5 |
| 2 | Similar cubes held for 3 days at 68°F and 60% relative humidity of the air | 2.3 | 32.7 |
| 3 | Specimen held in moist chamber for 28 days | 4.7 | 66.8 |
| 4 | A 2.82 x 2.94 x 2.99 in. specimen sawn out of pier brickwork, held in moist chamber for 28 days. | 6.0 | 85.3 |

The above table indicates that under no conditions did the mortar approach the compressive strength upon which the design was based (426 $lb/in^2$ according to NiTU - 120-55, Table 3).

2. Brick

Wall calculations called for Mark 100 brick. Tests on 10 bricks taken from the damaged brickwork showed that their compressive strength (111 $kg/cm^2$ - 1,580 lb/in ) was adequate, but that their bending strength (16 $kg/cm^2$ - 228 $lb/in^2$) corresponded to that of Mark 75 brick. Moreover, a number of hollow bricks with thinly sealed tops were discovered; the compressive strength of these was about the same as that of Mark 75 brick.

3. Peculiarities of Construction Work

a) Facing tile ends were embedded in brickwork bed joints to the depth of 10 cm. (3.94 in.); these joints were made some 3-4 cm. (1.18 - 1.58 in.) thick so that tile joints could be brought in line with the brickwork bed joints. This called for greater quantity of mortar (which was inadequate) and exercised corresponding weakening effect upon the brickwork.

b) To facilitate interior finishing work, heaters were installed in the hall and made operative some 2 weeks before the collapse. This brought about an intensive thawing of the walls on the inside. Since the weather happened to be very mild, thawing of the walls set in also on the outside. Thus, on the day of collapse, the mortar was at its minimum strength on both sides of the walls and frozen in the middle, where its compressive strength was somewhere between 2 and 10 $kg/cm^2$(28.4 - 142 $lb/in^2$).

c) The designers provided no chases in the wall for the piping leading to heating appliances. The plumbers' drawings, on the other hand, called for a 6.5 x 6.5 cm. (2.56 x 2.56 in.) horizontal chase in the wall right under the pier that was to fail. A day before the collapse, the plumbers cut the chase to the depth of from 12 to 14 cm. (4.72 - 5.51 in.) instead of the specified 6.5 cm. (2.56 in.). Effect of such a chase upon the strength of the brickwork even if it were laid under the summer conditions should have been verified by by new calculations. This was not done and the wall was allowed to be further weakened.

d) Neither the designers nor the builders provided for temporary bracing during the thaw period.

e) Pier reinforcement was laid incorrectly. This was assumed following the inspection of an undamaged pier.

f) It was noted that the piers remaining standing deviated outward from the perpendicular some 4.5 cm (1.77 in.) instead of the allowed 2 cm. (0.787 in.).

The above facts suggest that the inadequate bearing capacity of the pier during the thawing period constituted the main cause of the accident.

The main factors which affected the brickwork bearing capacity adversely were the following:

a) inadequate quality of mortar;

b) inadequate quality of brick;

c) incorrectly placed "zigzag" reinforcement;

d) absence of temporary bracing for the thawing period;

e) with brickwork laying calculated for summer conditions, the mortar mark was not raised one step to allow for the winter conditions which prevailed when the brick was actually laid;

f) weakening of the pier by the 12-14 cm. (4.74-5.51 in.) chase, not specified by the designers, as well as by the enlarged facing tile joints.

## Reconstruction

All piers that remained standing were strengthened by temporary outside supports.

The roof truss load was transferred to the foundations through steel supports which were anchored to the brick piers; this was done because the laboratory tests had indicated that under no conditions could the mortar in the piers reach the compressive strength envisaged by the designers.

## Note

The designers, assuming that the quality of materials would be adequate, calculated the pier stresses for summer conditions, presumably correctly. Their main fault lies in their allowing the erection of the walls by the freezing method without the necessary modification of the specifications calculated for summer conditions and in failing to design temporary bracing for the thawing period.

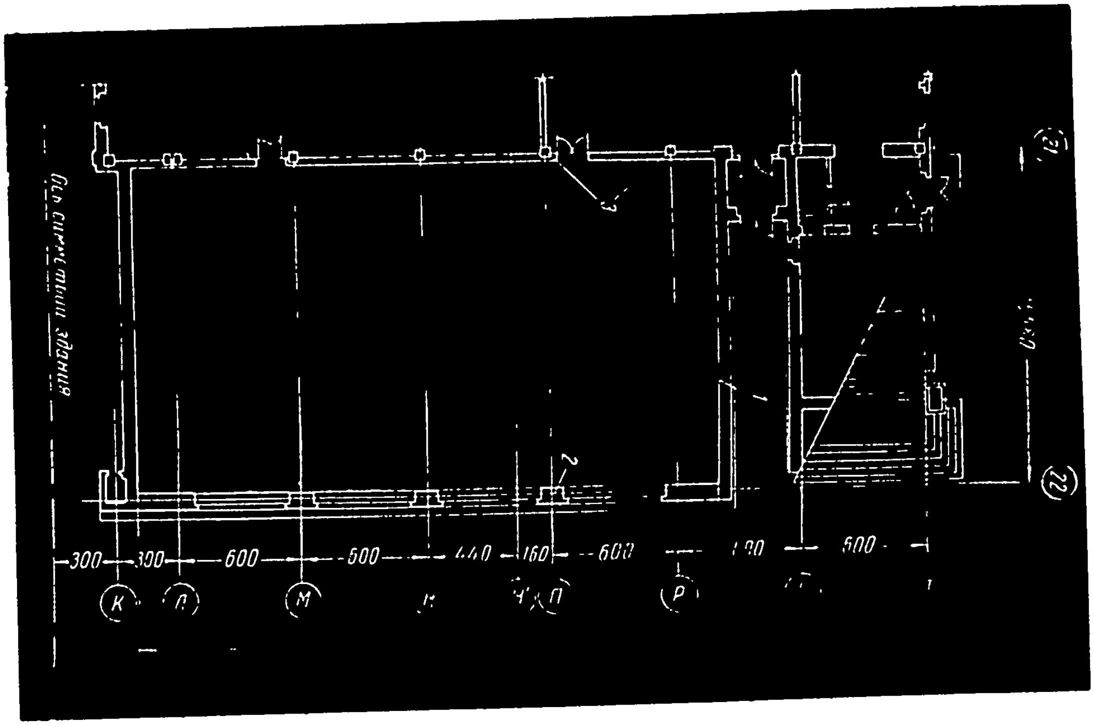
The building materials factories forwarded the wrong kind of brick to the building site under cover of "correct" certificates.

The builders used inadequate mortar without ever testing it and allowed further weakening of the pier by constructing faulty bed joints, incorrectly laying reinforcement and disregarding the effects of an unnecessarily deep chase cut by the plumbers.

In sum, the accident was due to negligence, primarily on the part of the designers and builders. The brick factories were at fault, too. However, the designers and builders should have tested the brick before and during the construction rather than after the accident.

## Source

Moscow TsINIS. Causes of Structural Failures, pp. 5-12.
TH 3401.M7




1. Training hall; 2. Collapsed pier; 3. Precast reinforced concrete columns

Plan

Conversion Table

| mm. | in. |
|---|---|
| 160 | 0.53 |
| 300 | 0.98 |
| 440 | 1.44 |
| 600 | 1.97 |
| 16,580 | 54.4 |
| 28,000 | 91.9 |

HAND GAMES TRAINING HALL, MOSCOW

Source: Moscow TsINIS. Causes of Structural Failures, p. 6. (TH 3401.M7)

PLATE 47

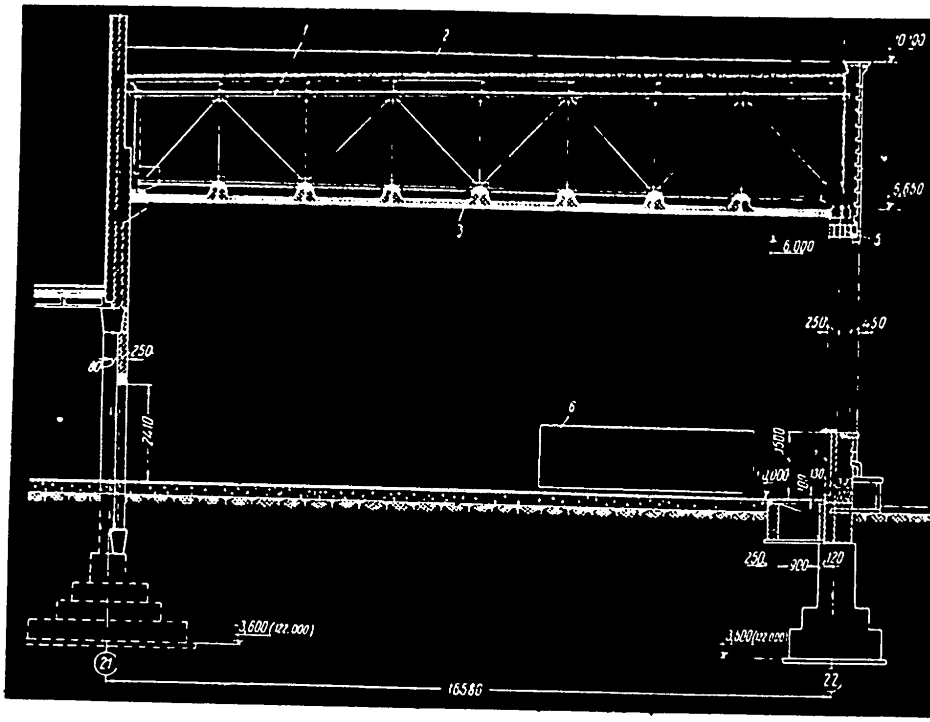



Рис 2 Поперечный разрез тренировочного спортивного зала

1—стальная ферма, 2—покрытие, 3—потолок; 4—облицовка из керамических плит; 5—сборная железобетонная перемычка; 6—ниши для панелей отопления; 7—ниша радиатора

1. Steel truss; 2. Roof slabs; 3. Ceiling; 4. Tile facing; 5. Precast reinforced concrete lintel; 6. Heating panel recess; 7. Radiator recess.

Transverse Section

Conversion Table

| mm. | ft. |
|---|---|
| 80 | 0.26 |
| 100 | 0.33 |
| 120 | 0.39 |
| 130 | 0.43 |
| 250 | 0.82 |
| 390 | 1.28 |
| 450 | 1.48 |
| 900 | 2.96 |
| 1,500 | 4.9 |
| 2,410 | 7.9 |
| 3,600 | 11.8 |
| 6,000 | 19.7 |
| 6,650 | 21.8 |
| 10,100 | 33.2 |
| 16,580 | 54.4 |

HAND GAMES TRAINING HALL, MOSCOW

Source: Moscow TsINIS. Causes of Structural Failures, p. 7 (TH 3401.M7)

PLATE 48

# CHAPTER XIII

## DEFECTIVE CONSTRUCTION OF CEMENT STORAGE SILOS

### Location

Bryansk cement factory.

### Structure

Two groups of freestanding reinforced concrete silos. Each group consists of six silos laid out as indicated in plan on Plate 49, fig. 1.

The silos provide storage space for cement produced by the Bryansk factory.

### Construction

The silos are filled with cement via an upper gallery; they are emptied pneumatically under air pressure of 3 atm. (44 lb/in$^2$) presumably through two bottom openings.

Each silo is 26.7 m. (87.6 ft.) high and has inside diameter of 9.5 m. (31.2 ft.). Its storage capacity is 2,600 m. tons of cement.

Foundations. Each silo is supported by nine columns, one at the center and eight along the circumference. These columns rest on a solid reinforced concrete slab which extends under all 12 silos.

Walls. Cylindrical silo walls are of Mark 140 reinforced concrete; they are 18 cm. (7.09 in.) thick.

Cross section of the wall is shown on Plate 49, fig. 3.

Wall reinforcement. Wall reinforcement consists of two rows of steel rods placed vertically and two horizontally. From the point of view of the distribution of horizontal reinforcement, the wall may be regarded as divided into 7 zones as follows:

| Zone No. | Height of the Zone mm. | ft. | Number of rods | Rod Diameter mm. | in. |
|---|---|---|---|---|---|
| 1 | 5,700 | 18.7 | 46 | 16 | 0.630 |
| 2 | 5,000 | 16.4 | 50 | 14 | 0.551 |
| 3 | 5,000 | 16.4 | 45 | 14 | 0.551 |
| 4 | 4,000 | 13.1 | 40 | 12 | 0.472 |
| 5 | 3,000 | 9.84 | 30 | 10 | 0.394 |
| 6 | 2,000 | 6.56 | 14 | 10 | 0.394 |
| 7 | 2,000 | 6.56 | 10 | 10 | 0.394 |

Vertical section of the wall showing reinforcement arrangement appears on Plate 49, fig. 2.



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Time of construction. Silos Nos. 4, 5, 6, 13, 14, 15 were built during 1951-1952; silos Nos. 7, 8, 9, 10, 11, 12 during 1952-1953.

## Collapse of Silo No. 7

On 11 November 1954, silo No. 7 collapsed in the following circumstances.

On that day, the silo filled to capacity (2,600 m.tons), was to be emptied for the first time since it had become operational. It was discovered in the attempt that the side opening as well as the two bottom openings of the silo were clogged. When one of the bottom openings had been cleared, 90 tons of cement were successfully unloaded in two stages into two railroad cars. Shortly afterwards, the silo collapsed with a roar.

First, at the height of from 8 to 10 m. (26.3 to 32.8 ft.), the upper part of the silo turned somewhat and began to list in the direction of silo No. 6; then, rapidly increasing vertical cracks appeared in its wall, which began to bulge; finally, the silo and its loading platform collapsed. Only a circular "stump" some 3.5 - 4.5 m. (11.5 - 14.8 ft.) high remained standing.

In their fall, the fragments of the collapsing silo made deep scars in the wall of the silo No. 12 and damaged silo No. 8, producing a 7 x 5 m. (23 x 16.4 ft.) gaping hole in its side at the height of some 5 m. (16.4 ft.).

A photograph of the wreckage is shown on Plate 50.

## Causes of Failure of Silo No. 7

Examination of the collapsed silo No. 7 disclosed that:

1) in places, the thickness of the wall was 16 cm. (6.30 in.) instead of the specified 18 cm. (7.09 in.)

2) the quantity of wall reinforcement in the 2nd zone (at the height of between 18.7 and 35.1 ft. from the bottom) amounted to 53% only of that specified.

It was concluded that:

1) the construction of the silo wall being defective, the immediate cause of the collapse might have been due to a sudden slippage inside the silo of some 2500 m. tons of packed cement into the cavity formed by the extrusion under air pressure of 90 m. tons of cement in the course of the emptying operation; (considerable air pressure, 44 $lb/in^2$ or more, applied in the lower part of the silo may have had a compressive effect on the upper layers; these packed layers may have formed for a brief period, in the course of emptying a sort of precariously stable cupola, particularly if cement was allowed to remain undisturbed in the silo for a considerable length of time);

2) the basic cause of the failure was due to the overstresses in the horizontal reinforcement developing under the action of operational loads.

(Note: The source makes no mention of possible imperfect roundness of the silo).

Defective Construction of the Remaining Silos

Collapse of silo No. 7 was followed by a very thorough examination of all the remaining silos. The quality of concrete was tested and the thickness of the walls verified. By means of specially cut grooves, wall reinforcement was checked at various levels as to the quality of the steel, the diameter of rods and the distance between the rods. The following conclusions were reached:

1) the quality of concrete and steel corresponded to that specified by the designers;

2) all walls had vertical cracks;

3) wall thickness (including that of silo No. 7) was 16 cm. (6.30 in.) in places instead of the specified 18 cm. (7.09 in.)

4) in emptying, the practice of gradually raising air pressure in silos through successive opening of air valves was not observed; there were not even pressure gauges on the air lines;

5) the silo design was not altogether correct although the designers had fully adhered to the "Technical Conditions" effective in 1950. The weaknesses of design were the following:

a) horizontal reinforcement was joined by means of lap joints without welding (such joints were accepted not only by the "Technical Conditions" of 1950 but also by the "Instructions on Calculations of Operational Loads in Silo Design," issued in 1952 under the title: U-115-52 MSPTI*);

b) pressure increase coefficient in Jansen's formula was taken in silo calculations to be equal 1.5; the U-115-52 MSPTI gives this coefficient as 2;

c) neither the U-115-52 MSPTI nor the designers had taken into account the effects upon reinforcement of the air pressure for forcing out the cement, which could be 44 $lb/in^2$, or higher in the case of clogged discharge openings.

6) the distance between the reinforcement rods was equal in places to 57 cm. (22.4 in.) instead of the specified 10-12 cm. (3.94-4.72 in.).

*MSPTI stands for: The Ministry for Construction of Heavy Industry Establishments.

Some defects of individual silos, particularly with respect to proper distribution of reinforcement appear in the following table:

| Silo No. | Principal Defects |
|---|---|
| 4 | The quantity of reinforcement in the lower half of the silo amounted to 89% of that specified. |
| 8 | Numerous vertical cracks up to 12 mm. (0.472 in.) wide; wall bulged in the direction of side openings; reinforcement, distributed very unevenly, amounted to 57% of that specified. |
| 10 | Three test grooves at the height of 5.32 m. (17.4 ft.) from the bottom indicated that reinforcement amounted to from 49 to 58% of that specified. |
| 12 | Reinforcement in places amounted to no more that 40% of that specified; wall bulged in the direction of side opening; many vertical cracks - cracks were noted as early as June 1954, but they were only patched up with cement. |
| 13 | Reinforcement in the first zone of the wall amounted to 92%, and in the second zone to 67%, of that specified. |

## Reconstruction

Silos Nos. 8, 9, 10, 11, 12 were put temporarily out of commission until such time as they could be appropriately strengthened.

Silos Nos. 4, 5, 6, 13, 14, 15 were to be utilized up to the height of 20 m. (65.6 ft.) only, i. e. to about 3/4 of their capacity.

Vertical and cross sections of the reconstructed Silo No. 7 are shown on Plate 51, figs. 1, 2, and 3.

Silo No. 7 was reconstructed as follows. The outside surface of the remaining "stump" was notched, washed and surrounded to the height of 3.5 m. (11.5 ft.) from the bottom by a reinforced concrete casing 15 cm. (5.91 in.) thick. Above the casing, the wall was reconstructed in accordance with the old dimensions (inside radius - 31.2 ft., wall thickness - 7.09 in.). Reinforcement was handled in the light of experience obtained in connection with the failure of silo.

Strengthening of other silos presented an urgent problem lest the factory production be hampered. For this purpose, casings made of welded steel sheets were erected around the entire height of each silo. The space between the outside surface of silo walls and the casing was filled with concrete; the casings were painted with aluminum paint.

A photograph of silos in the process of being strengthened is shown on Plate 52.

## Note

Responsibility for defective construction rests with the designers and builders. For once, the quality of construction materials does not seem to have played any part in the failure of the structure.

## Source

Moscow TsINIS. Causes of Structural Failures, pp. 42-49. TH 3401.M7

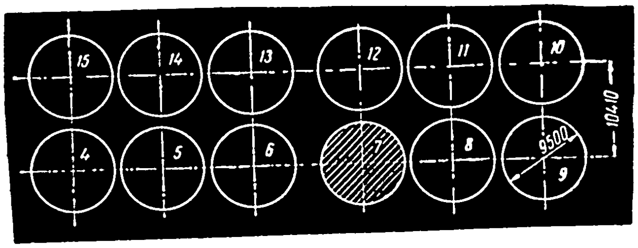
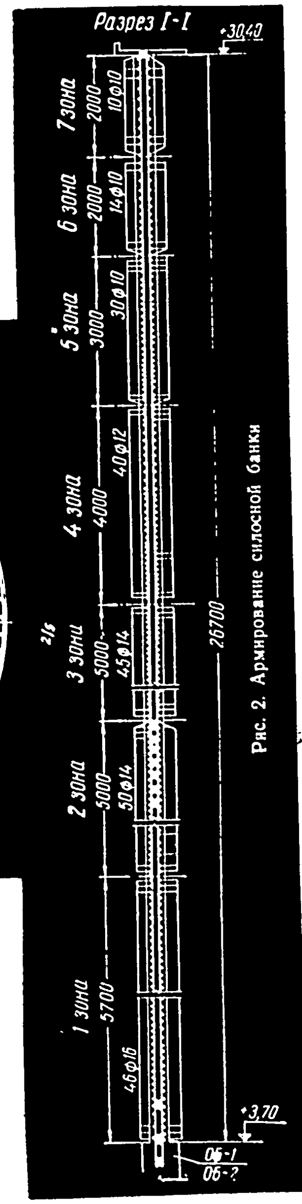



Fig. 1. Silo layout




**Reinforcement Conversion Table**

| mm. | in. |
|---|---|
| 10 | 0.394 |
| 12 | 0.472 |
| 14 | 0.551 |
| 16 | 0.630 |

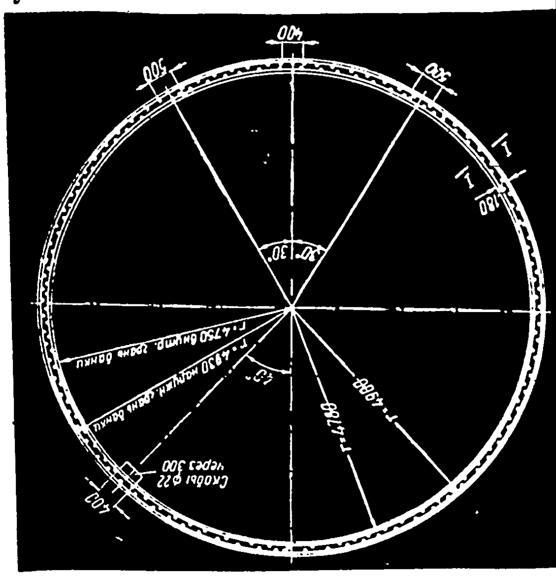


Fig. 3. Silo Wall; cross section.

**Conversion Table**

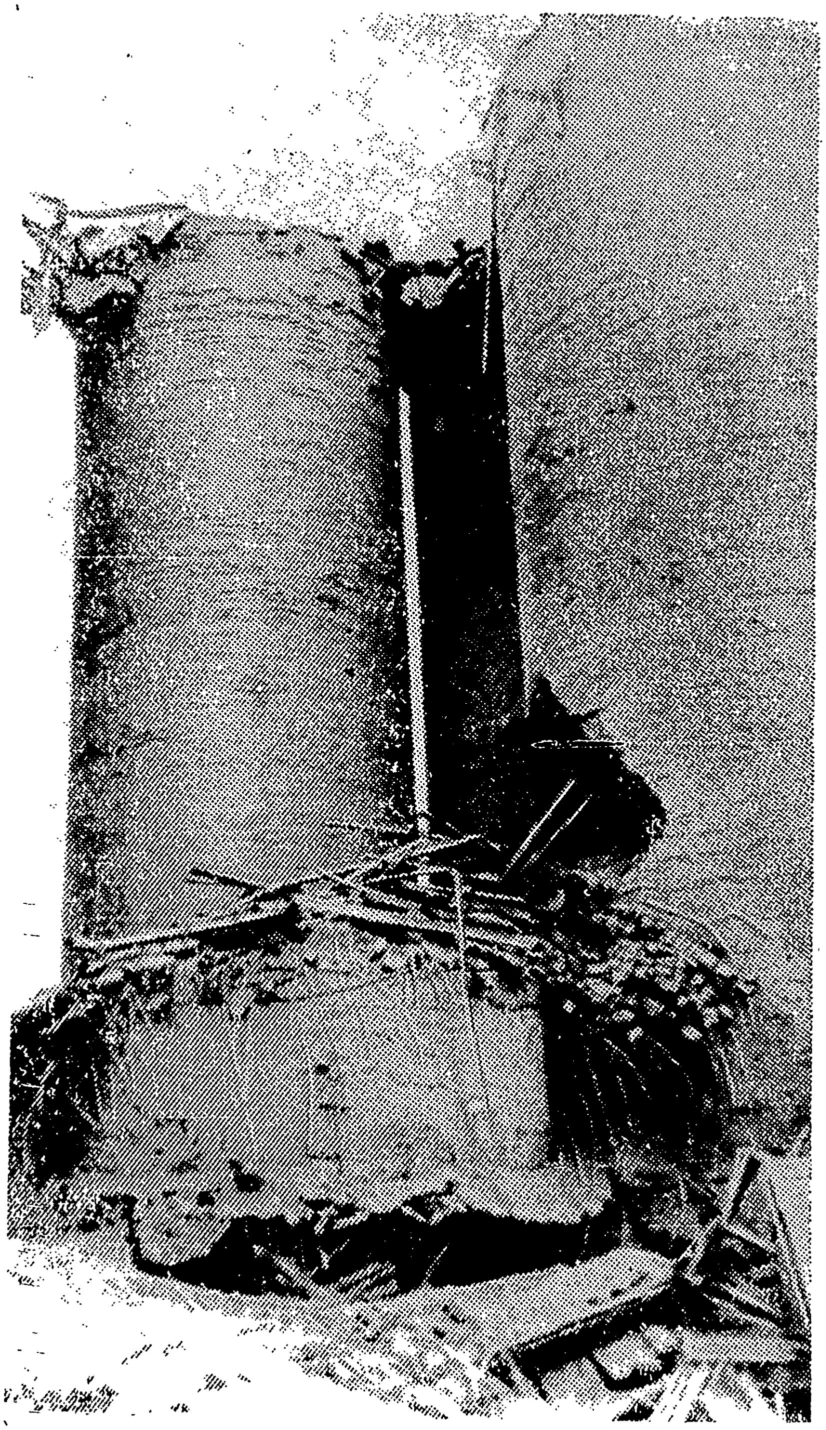
| mm. | ft. |
|---|---|
| 180 | 0.59 |
| 400 | 1.31 |
| 500 | 1.64 |
| 2,000 | 6.56 |
| 3,000 | 9.84 |
| 4,000 | 13.1 |
| 4,780 | 15.7 |
| 4,900 | 16.1 |
| 5,000 | 16.4 |
| 5,700 | 18.7 |
| 9,500 | 31.2 |
| 10,410 | 34.2 |
| 26,700 | 87.6 |

Fig. 2. Silo wall; vertical section showing reinforcement distribution in 7 zones.

DEFECTIVE CEMENT STORAGE SILOS (Bryansk factory)

Source: Moscow TsINIS. Causes of Structural Failures pp. 42-43. (TH 3401.M7)

PLATE 49



SILO NO.7 AFTER THE COLLAPSE (Bryansk factory)
Source: Moscow TsINIS. Causes of Structural Failures, p. 44. (TH3401.M7)

PLATE 50

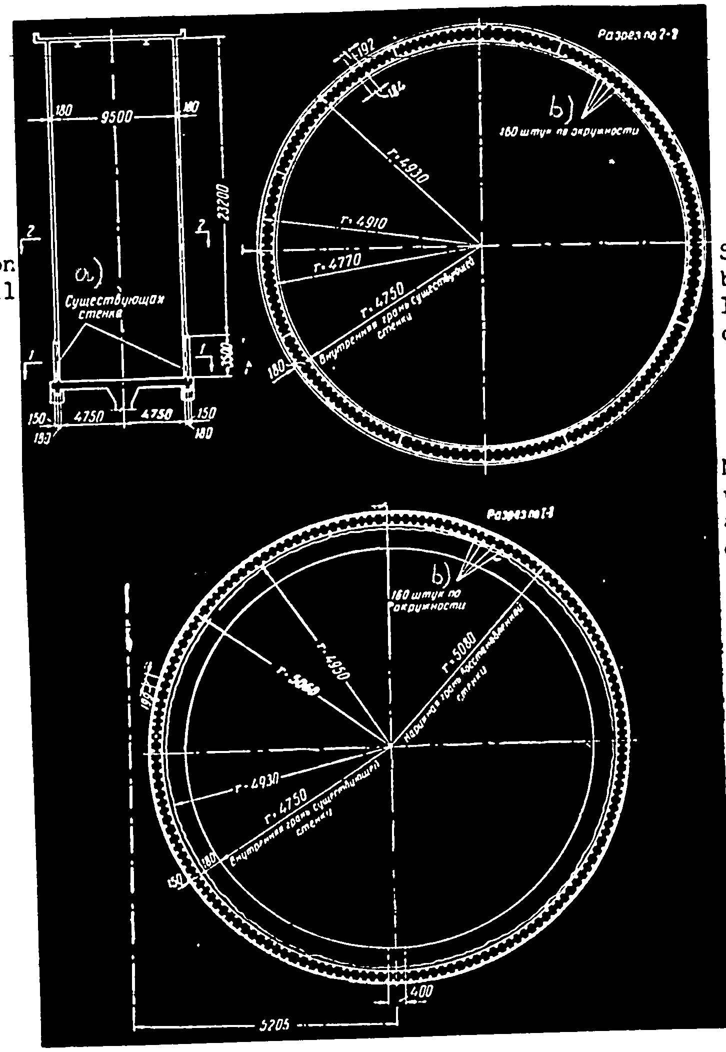



Fig. 1
Vertical Section
a) The old wall

Fig. 2
Section 2-2. New wall.
b) Reinforcement;
160 vertical bars along circumference.

Note: In the plans, the wall thickness is shown at about twice the scale of the over-all dimensions

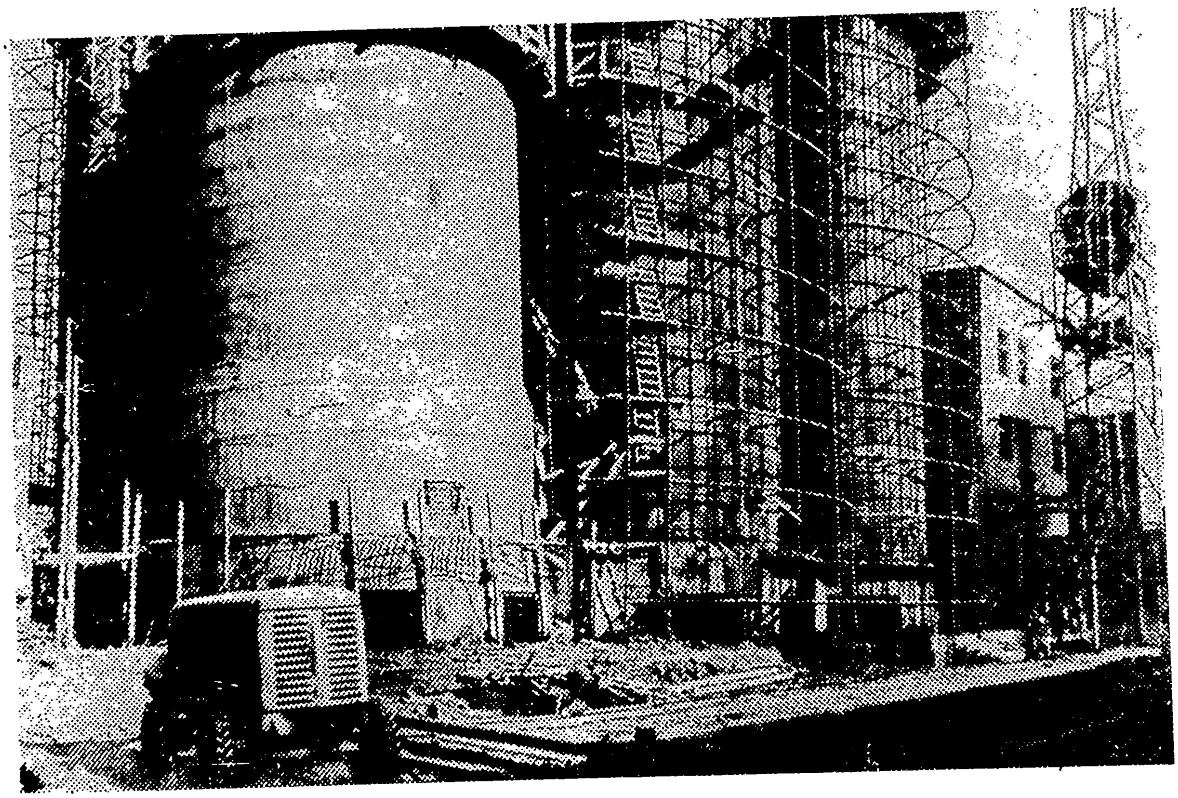
Fig. 3. Section 1-1. Encased old wall.
b) Reinforcement; 160 vertical bars along circumference.

Conversion Table

| mm. | ft. | mm. | ft. |
|---|---|---|---|
| 150 | 0.49 | 4,910 | 16.1 |
| 180 | 0.59 | 4,930 | 16.2 |
| 192 | 0.63 | 4,950 | 16.3 |
| 199 | 0.65 | 5,060 | 16.6 |
| 400 | 1.31 | 5,080 | 16.7 |
| 3,500 | 11.5 | 5,205 | 17.1 |
| 4,750 | 15.6 | 23,200 | 76.1 |
| 4,770 | 15.7 | | |

RECONSTRUCTION OF SILO No. 7 (Bryansk factory)
Source: Moscow TsINIS. Causes of Structural Failures, p. 48. (TH 3401.M7)

PLATE 51



STRENGTHENING OF SILOS (Bryansk factory)
Source: Moscow TsINIS. Causes of Structural Failures, p. 49. (TH 3401.M7)

PLATE 52

# CHAPTER XIV

## FAILURE OF A REINFORCED CONCRETE SETTLING TANK (DOMODEDOVO)

### Location

Purifying plant, Domodedovo, Podolsk District, Moscow Oblast'.

### Structure

Circular reinforced concrete settling tank with conical bottom, sunken in the earth.

### Construction

Plan and section of the tank are shown on Plate 53, figs. 1 and 2.

The tank is 11 m. (36.1 ft.) high; its inside diameter is 9 m. (29.5 ft.). In its upper part, the tank has 2 reinforced concrete settling troughs with reinforced concrete distributing and collecting pans.

A reinforced concrete silt chamber is connected to the upper part of the tank with a monolithic joint. The overall dimensions of the chamber are:

| | |
|---|---|
| Length | 2.8 m. (9.18 ft.) |
| Width | 2.1 m. (6.89 ft.) |
| Height | 2.65 m. (8.69 ft.) |
| Wall thickness | 30 cm. (11.8 in.) |

Foundation. The bottom of the tank rests on an inverted conical foundation of Mark 50 concrete; it appears to be about 7 inches thick.

Conical Tank Bottom. It is of Mark 140 concrete; 12 cm. (4.72 in.) thick.

Tank Wall. The circular tank wall is of Mark 140 concrete. It is 12 cm. (4.72 in.) thick in its upper part and 20 cm. (7.87 in.) in its lower part. The part adjoining the bottom has, on the outside, an added thickness of 10 cm. (3.94 in.) extending to the height of 70 cm. (27.6 in.) and, on the inside, an angle bracket built to the same height all along the circumference.

Reinforcement. Vertical reinforcement of the wall and reinforcement of the bottom consist of smooth steel rods. Horizontal reinforcement and that of the angle bracket is made of hot-rolled deformed bars. The wall has double row of reinforcement to the height of 4 m. (13.1 ft.); above that height reinforcement is arranged in a single row.

Insulation. The wall was to be insulated on the outside with moisture resistant material to the height of 5.82 m. (19.1 ft.) and protected to the same height with a brick wall. This wall was to be one stretcher thick and laid with cement Mark 50 mortar.

Coating. According to the specifications, the interior surfaces of the tank, troughs, and pans were to be gunited.

## Construction Procedure and Failure of the Tank

Pouring was started in August 1955.

The inner part of the form was built to the entire height of the tank; the outer part was erected by circular sections, 1 m. (3.28 ft.) high, as the pouring advanced.

The filling of one circular section with concrete was sometimes done in one operation; sometimes in 2 or 3 stages with interruptions of from 16 to 20 hours or even 2 or 3 days.

Concrete was brought in part from the Domodedovo factory for concrete products; in part was prepared at the site. In either case, no certificate of quality of concrete was made. No laboratory test cubes were prepared, nor was the work progress log kept.

Pouring was finished in November 1955.

Removal of interior concrete form disclosed that the inner surface of the wall was honeycombed particularly in its lower part. A piece of concrete was easily broken off of that part and sent to the laboratory. Its strength was so low that no test of mechanical properties could be made.

In these circumstances, the inner surfaces were not gun sprayed - instead, they were plastered. The outer concrete form was not dismantled; consequently, no insulation could be applied nor the brick wall built. The cavity around the tank and silt chamber, their concrete forms still in place, was filled with soil moved by a bulldozer.

Shortly afterwards, bulging and cracks appeared on the surface of the tank wall in the region of its monolithic joint with the silt chamber; a gap formed between the chamber and the tank wall. The chamber was settling fast. To remedy the situation, reinforcement between the chamber and the tank was cut; bricks were laid in the gap and plastered.

It was in this condition that the operating agency accepted the tank for temporary use in December 1955.

With the spring of 1956, came further settling of the fill around the tank. The chamber sank lower; the pans sagged in places from 30 to 50 cm. (11.8 - 19.7 in.) and broke.

The purifying plant had to be shut down.

Water was never pumped out of the tank; it soon disappeared, however, only some of it remaining in the conical bottom. It was then that the discovery was made that at the height of 1.5 - 2 m. (4.92 - 6.56 ft.) from the bottom the tank wall crumbled all along the circumference revealing bare and bulging reinforcement. The part of the wall above the damaged section slid down some 20-30 cm. (7.87-11.8 in.). Section of the damaged tank is shown in drawing on Plate 54.

## Causes of Failure

The failure of the tank was primarily caused by the low quality of concrete used in its construction, which was apparently due to two factors:

1) it appears that unwashed fine sand with high content of clay and other impurities had been used in the preparation of concrete.

2) the builders disregarded Section III, Concrete and Reinforced Concrete Jobs, Paragraph 159 of the Soviet "Technical Conditions" for the execution and acceptance of construction and erection jobs. Section III requires that following an interruption of more than 2 hours the pouring of concrete may be resumed only after the previously poured concrete had reached the compressive strength of not less than 12 kg/cm$^2$ (170 lb/in$^2$).

This procedure involves laboratory testing which was disregarded. Equally disregarded was the practical rule to the effect that concrete made with portland cement of Mark lower than 400 and poured in September-October requires about 100 hours to reach the strength of 12 kg/cm$^2$ or 170 lb/in$^2$.

## Reconstruction

A new tank was built. The walls of the old tank were used as external concrete form.

Settling troughs were cut off of the old tank and joined monolithically with the walls of the new one.

Collecting and distributing troughs as well as the silt chamber were built anew.

## Note

Neither the quality of the uncertified concrete supplied by the Domodedovo factory of reinforced concrete objects nor the quality of concrete prepared at the site were tested by the builders.

During the earth-filling operation of the cavity around the tank supports of the silt chamber were disturbed, apparently by the bulldozer, before the silt chamber concrete was allowed to set.

The rules governing the pouring of concrete after an interruption were not followed. This naturally affected the structure of the previously poured and not yet sufficiently hardened concrete.



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The above facts testify to negligence on the part of the builders who seem to be solely responsible for the faulty construction.

Source

Moscow TsINIS. Causes of Structural Failures, pp. 33-38.
TH 3401.M7

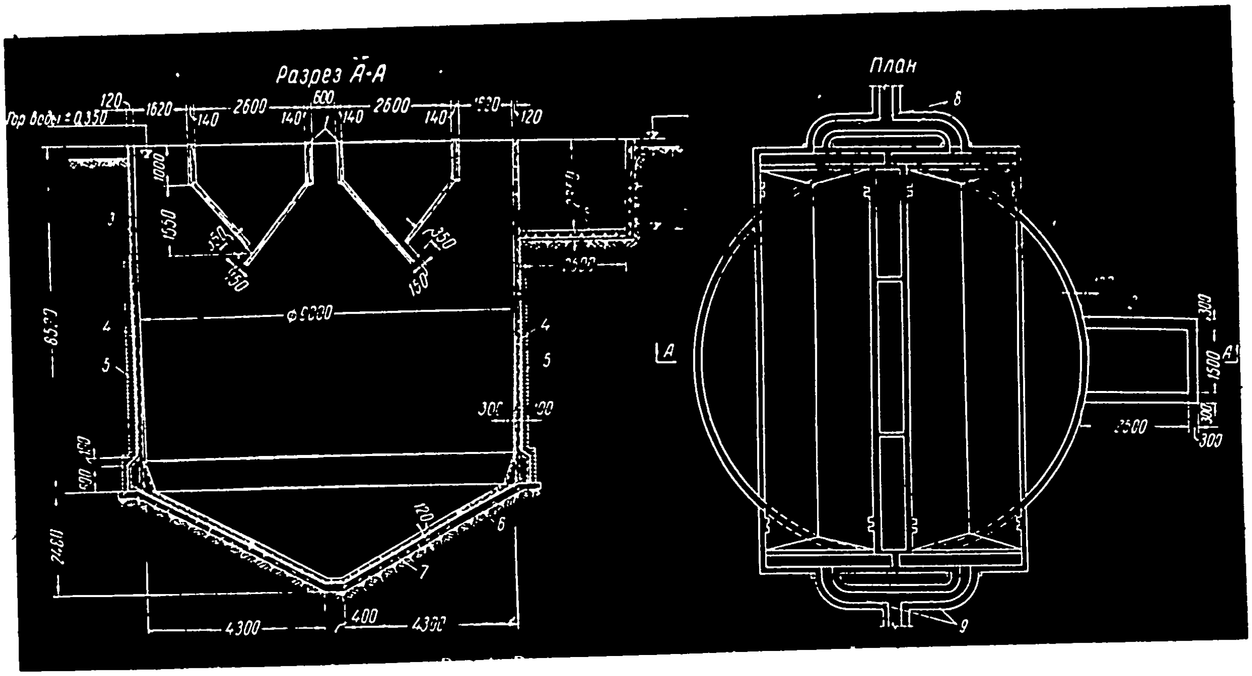


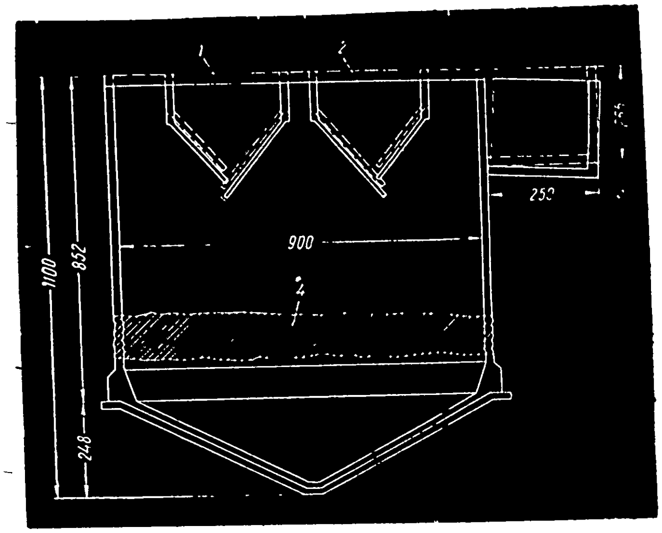

Fig. 1. Section A-A Fig. 2. Plan
1. Settling trough; 2. Silt chamber; 3. Tank wall; 4. Water insulation;
5. Brick protecting wall; 6. Conical tank bottom; 7. Mark 50 concrete foundation;
8. Collecting pans; 9. Distributing pans

Conversion Table

| mm. | ft. | mm. | ft. |
|---|---|---|---|
| 100 | 0.33 | 1,550 | 5.09 |
| 120 | 0.39 | 1,600 | 5.25 |
| 140 | 0.46 | 1,620 | 5.32 |
| 150 | 0.49 | 2,350 | 7.72 |
| 300 | 0.98 | 2,480 | 8.15 |
| 350 | 1.15 | 2,500 | 8.21 |
| 400 | 1.31 | 2,600 | 8.54 |
| 600 | 1.97 | 4,300 | 14.1 |
| 1,000 | 3.28 | 8,520 | 28.0 |
| 1,500 | 4.93 | 9,000 | 29.5 |

REINFORCED CONCRETE SETTLING TANK (Domodedovo)
Source: Moscow TsINIS. Causes of Structural Failures, p. 34. (TH 3401.M7)

PLATE 53




1. Tank level at erection; 2. Tank level after wall disintegration; 3. Silt chamber after settlement; 4. Crumbled zone of the wall.

View of the Damage

**Conversion Table**

| cm. | ft. |
|---|---|
| 248 | 8.15 |
| 250 | 8.21 |
| 265 | 8.70 |
| 852 | 28.0 |
| 900 | 29.5 |
| 1,100 | 36.1 |

REINFORCED CONCRETE SETTLING TANK (Domodedovo)

Source: Moscow, TsINIS. Causes of Structural Failures, p. 37 (TH 3401.M7)

PLATE 54

# CHAPTER XV

## RACKING OF A STEEL BUILDING FRAME (SIBERIA)

### Location

Somewhere in Siberia

### Structure

Four-aisle steel building frame without roof covering. Upon completion, the structure is to house the second section of a Heat and Electric Power Station (TETs).

### Soil at the Building Site

The structure is erected on a non-settling, water-saturated loess-like loam layer.

### Construction

Plan and veritical section of the structure are shown on Plate 55, fig. 1.

Dimensions:

| | |
|---|---|
| Overall width | 68.5 m. (224.7 ft.) |
| Overall height | 40.2 m. (131.9 ft.) |
| Overall length | 45.0 m. (147.6 ft.) approx. |
| Bay length | 6.2 m. ( 20.3 ft.) |

Foundations. Foundations are continuous, with reinforced concrete pillars under columns. Along axes B and C (bunker section) the columns rest on a solid reinforced concrete slab which is 1 m. (3.28 ft.) thick.

Construction Time Table. The steel frame was erected in the middle of 1952. Before the joints of the frame were permanently fastened, the columns were aligned in July 1952. Sights were taken of the tops of the anchor bolts of the column shoes. Only a few columns were found to be out of line; they had shifted southeastward from 5 to 20 mm. (0.197 to 0.787 in.) beyond the allowable limit.

### Signs of Racking

In February 1953, when the filling of frames with brickwork was in progress, it was noted that at the height of approximately 19 m. (62.3 ft.) along axis B the brickwork was outside the plane of the frame. This suggested that the columns might have been out of line some 60-90 mm. (2.36-3.54in.) beyond the allowable limit.

Investigation established that:

1) All columns were out of plumb and leaned in the south-easterly direction. The joints at the height of 33 m. (108 ft.) along axes C and D, for instance, had shifted horizontally from 100 to 153 mm. (3.94 to 6.0 in.)

2) The maximum shift of columns in both longitudinal and transverse directions occurred along axes B and C in rows from 2 to 6 and decreased somewhat toward row 8, i. e., toward the end wall of the operational part of the TETs. (Plate 55, fig. 2).

3) Foundations had moved upwards and so did the columns up to 60 mm. (2.36 in.) in rows 2-6 along axis B with the heaving gradually decreasing toward row 8 along axes A and B (Plate 55, fig. 2).

4) The soil around foundations was frozen along Axis B to a depth of 40 cm. (15.8 in.) and along axis C to a depth of 15 cm. The soil did not lose its ductility. It was permeated with ice flakes; brought into a warm room it acquired the consistency of thick sour cream within 20 minutes. A diagram showing vertical shifting of foundation in the frozen ground is shown on Plate 55, fig. 3.

Foreseeing the possibility of freezing of foundations, before heavy frost, the builders packed some 1,000 m$^3$ (1,308 yd$^3$) of boiler slag around the foundations. However, they left uncovered large areas located between rows 2-5 in the reinforced concrete slab underlying aisle B-C.

## Causes of Racking

The heaving of the frame was due to a non-uniform freezing of the soil under foundations, particularly under the solid slab of B-C aisle.

In general, the most dangerous source of unexpected forces is the upper 70 cm. (27.6 in.) layer of frozen bulging soils. In the case under consideration, a 40 cm. (15.8 in.) frozen layer of a particular soil heaved the structure some 60 mm. (2.36 in.); this corresponds to an increase of 15 % in the volume of the soil.

## Plumbing the Structure

It was out of the question to wait for the natural settlement of foundations, because of rigid construction deadlines.

Electrical or steam heating of the frozen soil under the 3-foot thick reinforced concrete slab was rejected as impracticable.

The method finally adopted is indicated in the sketch on Plate 55, fig. 4.



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Three 2-inch perforated steel pipes were laid along the length of the slab in aisle B-C and connected with live steam pipes of the TETs. Excavation in aisle A-B and the "trough" B-C were filled with water. A 1,000 $m^2$ (10,760 $ft^2$) "pool" was thus formed. A temperature of some 60-70°C (140-158°F) was maintained in the "trough" by the live steam passing through the perforated pipes. The temperature of water in aisle A-B stood at 5-10°C (41-50°F) without heating.

Foundation heating operation was started on 25 March and ended on 6 April 1953. Observations indicated that on 11 April 1953, the greater part of foundations had passed the point of the beginning of heaving and that the natural settlement of foundations was beginning.

Perfect plumbing of columns was never achieved, but the linear shift of the columns was considerably reduced. The following table gives an example of the decrease in linear shift of a frame joint at the height of 108 ft. along axis D:

| Row | Ultimate Shift | | Shift decreased to | |
|---|---|---|---|---|
| | mm. | in. | mm. | in. |
| 1 | 78 | 3.07 | 25 | 0.984 |
| 2 | 112 | 4.41 | 15 | 0.591 |
| 3 | 153 | 6.02 | 50 | 1.97 |
| 4 | 112 | 4.41 | 40 | 1.57 |
| 5 | 131 | 5.12 | 90 | 3.54 |
| 6 | 74 | 2.91 | 40 | 1.57 |
| 7 | 73 | 2.87 | 40 | 1.57 |

With settlement of foundations stablized, those parts of the structure were strengthened where deviations from "The Technical Conditions" were observed. No further complications were encountered.

Source

Stroitel'naya Promyshlennost', No. 3, 1955, pp. 24-26.

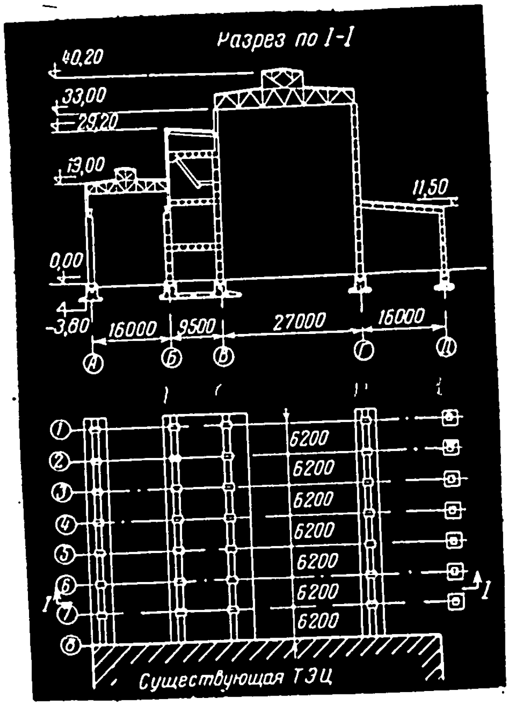



Fig. 1. Plan and vertical section; the frame adjoins the TETs already in operation.

**Conversion Table**

| mm. | ft. |
|---|---|
| 6,200 | 20.4 |
| 9,500 | 31.2 |
| 16,000 | 52.5 |
| 27,000 | 88.6 |

| m. | ft. |
|---|---|
| 3.80 | 12.5 |
| 11.50 | 37.8 |
| 19.00 | 62.4 |
| 29.20 | 95.8 |
| 33.00 | 108.3 |
| 40.20 | 132.0 |

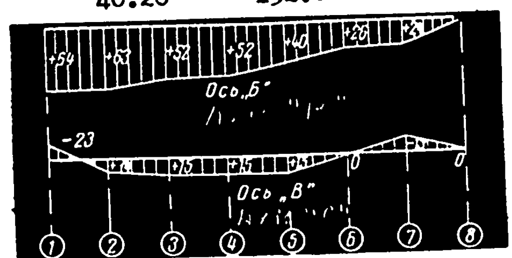



Fig. 3. Heaving of foundations along axes B and C. (displacements in mm.).

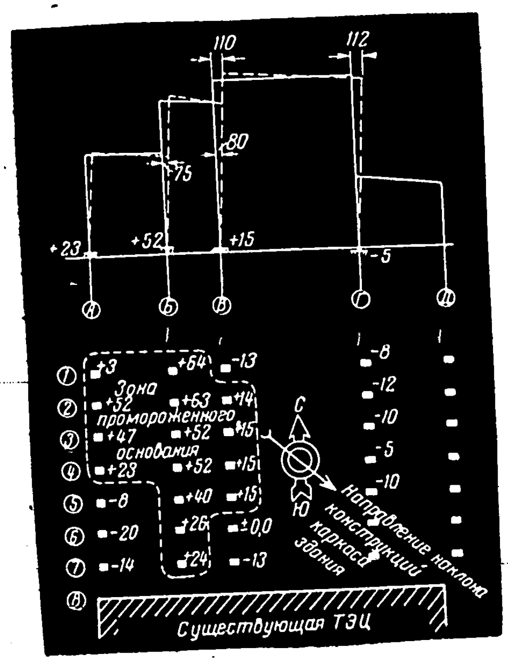



Fig. 2. Heaving of foundations and racking of structure; frozen area under foundations is enclosed by dotted line (displacements in mm.)

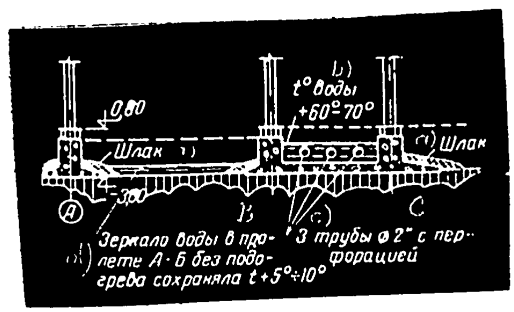



Fig. 4. Method of plumbing the structure; a) slag; b) water temp. 140-158°F; c) 3 perforated 2" pipes; d) water without heating, temp. 41-50°F.

RACKED STEEL FRAME OF A TETs ( Siberia)

Source: Stroitel'naya Promyshlennost', No. 3, 1955, pp. 24-26.

PLATE 55

# CHAPTER XVI

## PARTIAL COLLAPSE OF THREE-HINGED TRAPEZOIDAL STEEL BENTS

### Location

The Middle Ural Region

### Structure

Three-hinge trapezoidal arch steel bents. The structure which has been in operation since 1951 serves as a raw material (unspecified) storehouse for an industrial plant.

### Construction

Plan of the structure is shown on Plate 56, fig. 1.

The structure is 234 m. (767. 6 ft.) long; two trarsverse expansion joints divide it in 3 sections; north, middle and south.

The trapezoidal arches are constructed of I-45 steel beams (no weight is given; refer to SES Report No. 1., Tables: 1.0256B and 1.0257B).

| | |
|---|---|
| Arch span | 30 m. (98.4 ft.) |
| Bay length | 6 m. (19.7 ft.) |
| Walls | one stretcher, slag concrete. |
| Roof | purlins covered with corrugated asbestos board. |
| Monitor | longitudinal (Plate 56, fig. 2) |
| Conveyer gallery | longitudinal, located in the middle of the aisle at the height of 14 m. (45.9 ft.) |

Arches along axes 29, 30, and 31 (Plate 56, fig. 1) were strengthened in connection with a re-loading installation located underneath (no details are specified).

### Partial Collapse of the Roof

At the end of December 1956, with the outside temperature of -13°C (8.6°F), the middle part of the roof collapsed.

According to witnesses, the failure started with axis 28 and spread northward with western semi-arches falling first and being followed by the eastern. The strengthened arches along axes 29, 30, and 31 were damaged but remained standing.

The first to fall and the most warped western semi-arch along axis 28 is shown in photograph on Plate 57.

## Causes of Collapse

The structure was designed in accordance with OST-90058-40 norms wherein the allowable snow load value of 60 $kg/m^2$ (12.3 $lb/ft^2$) was indicated for the slope of the roof ($38^{o}40'$) in the case under consideration.

Following the collapse of the roof, the depth of snow on its remaining parts was actually measured. It was found that the snow layer was:

| | | | |
|---|---|---|---|
| | 150-200 mm. | (5.91-7.87 in.) | thick on the conveyer gallery roof; |
| | 570 mm. | ( 22.4 in.) | thick on the slope of the arch (perpendicular to it); |
| up to | 1,100 mm. | ( 43.3 in.) | thick on the monitor (west side). |
| up to | 900 mm. | ( 35.4 in. ) | thick on the monitor (east side) |

Photographs of the snow load on the damaged roof are shown on Plate 58.

The snow was heavily mixed with dust and ash, its volume weight was 235 $kg/m^3$ (14.6 $lb/ft^3$). Presence of considerable amount of ash in the snow not only increases the volume weight of the snow but also raises the value of the coefficient of friction between the snow and the roof.

The actual snow load on the sloping part of the roof was equal to 134 $kg/m^2$ (27.4 $lb/ft^2$), i. e. more than twice the allowable value. Recalculations on the above basis indicated that stresses in the steel of the arch had passed the yield point. Hence the buckling and the collapse.

In this connection, it may be noted that the total precipitation in the form of snow in this region, as given by the meteorological reports, is the following:

| Year | Snowfall | |
|---|---|---|
| | mm. | in. |
| 1951/52 | 74.4 | 2.92 |
| 1952/53 | 129.5 | 5.10 |
| 1953/54 | 102.1 | 4.02 |
| 1954/55 | 134.8 | 5.31 |
| 1955/56 | 85.9 | 3.38 |
| 1956 (to 26 Dec.) | 147.0 | 5.79 |

## Source

Stroitel'naya Promyshlennost', No. 7, July 1957, pp. 18-19.

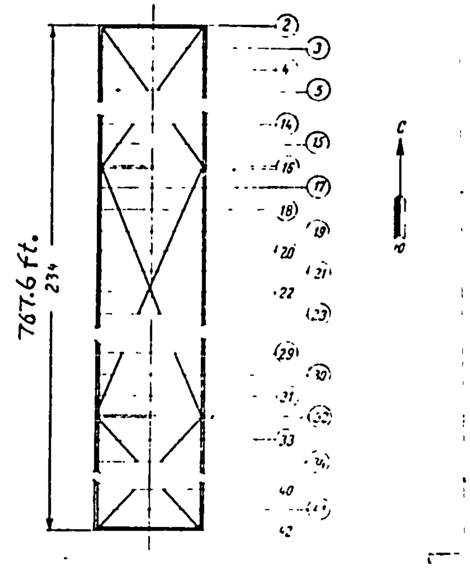



Fig. 1. Plan

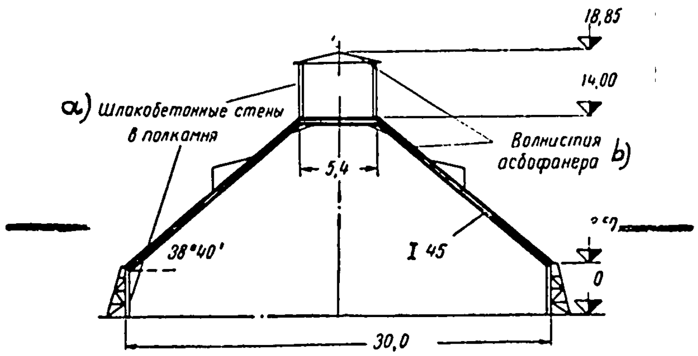


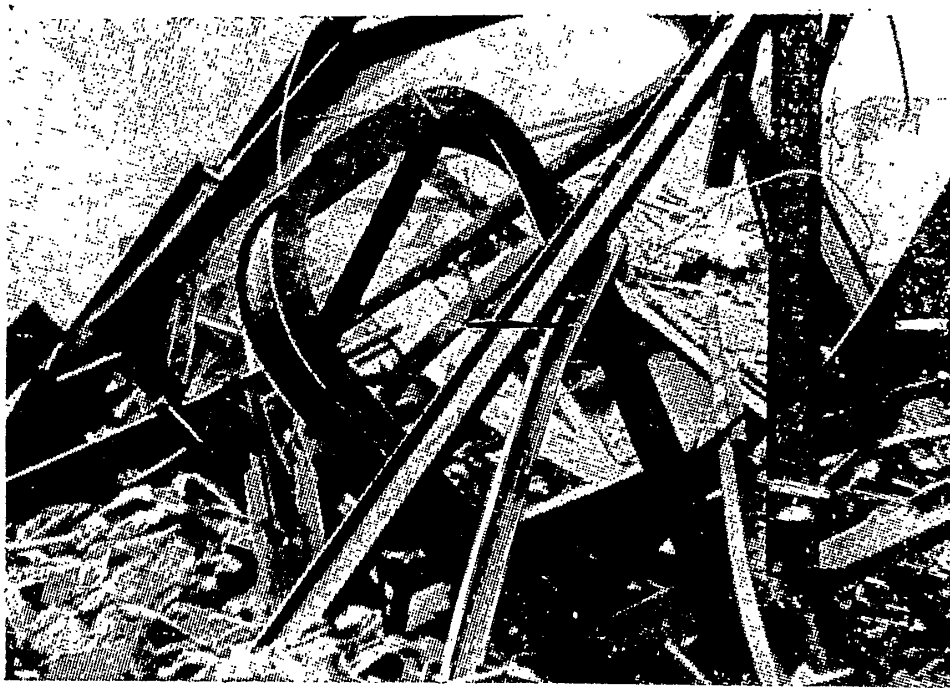

Fig. 2. Transverse section.
a) one-stretcher slag concrete walls
b) corrugated asbestos panels

Conversion Table

| m. | ft. |
|---|---|
| 3.50 | 11.5 |
| 5.40 | 17.7 |
| 14.00 | 46.0 |
| 18.85 | 61.9 |
| 30.00 | 98.4 |
| 234.00 | 767.6 |

THREE-HINGED TRAPEZOIDAL STEEL BENTS
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 19

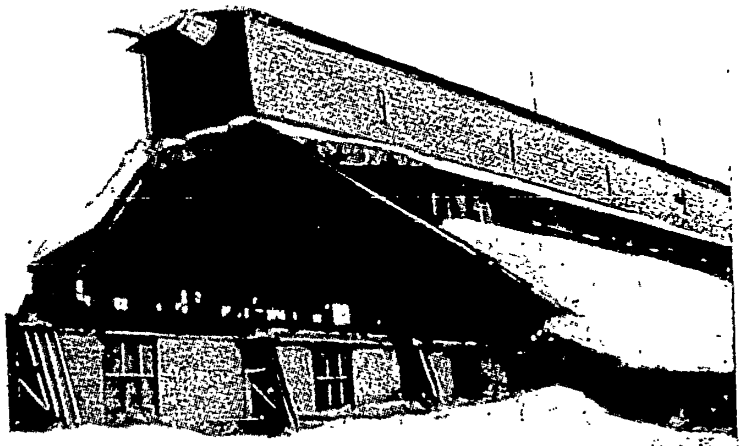
PLATE 56

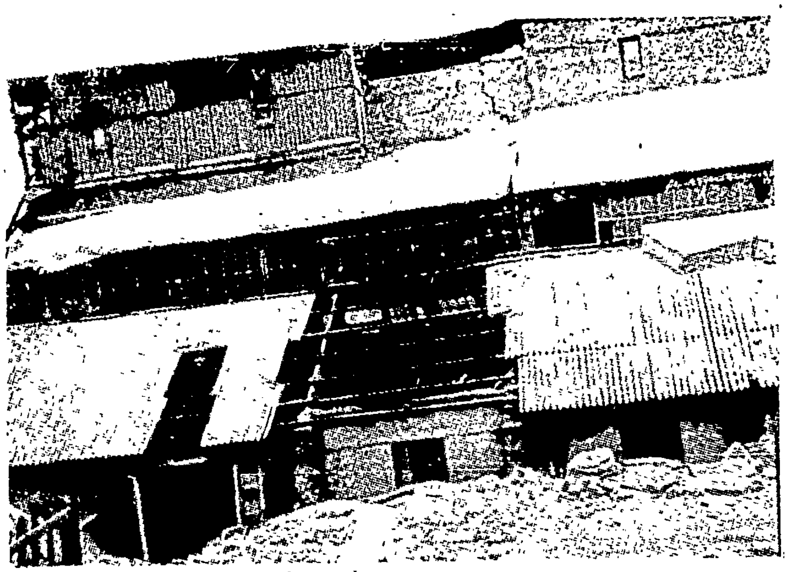


Western semi-arch (axis 28) after collapse

THREE-HINGED TRAPEZOIDAL STEEL BENTS
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 19

PLATE 57





Views of snow load on the damaged roof

THREE-HINGED TRAPEZOIDAL STEEL BENTS
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 20

PLATE 58



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# CHAPTER XVII

## FAILURE OF TWO STEEL ROOF TRUSSES IN AN INDUSTRIAL STRUCTURE

### Location

The Middle Ural Region

### Structure

Two-aisle industrial structure with Warren steel roof trusses and a longitudinal monitor. Wall construction is not indicated.

### Construction

The steel roof truss, span - 24 m. or 78.7 ft., is shown in drawing on Plate 59.

Roof. The steel roof trusses are covered with:

a) extra strong corrugated asbestos board;
b) 2 layers of slag wool with total thickness of 90 mm. (3.54 in.);
c) 1 layer of cement 35-40 mm. (1.38-1.57 in.) thick;
d) water-proofing material

### Failure of Two Steel Roof Trusses

Early in 1957, web diagonals No. 8 bulged out in two trusses in the leeward aisle of the structure. A 600 mm. (23.6 in.) buckling occurred in the lower joints of the damaged diagonals. The roof trusses did not collapse, part of their load being borne by the monitor.

### Causes of Truss Damage

It was established that:

1. The snow on the roof was mixed with industrial dust; its volume weight varied from 220-390 $kg/m^3$ (14-24 $lb/ft^3$); actual snow load was equal in places to 500 $kg/m^2$ (102 $lb/ft^2$), i.e., more than 5 times the allowable load; however, since the shape of the snow layer was almost triangular, the total snow load was only slightly above the allowable.

2. The total load on the roof truss (dead and snow) was 51.73 m. tons (114 kips) as against the allowable load of 36.6 m. tons (80.7 kips). In the original load calculations, the normal force and stress in the eventually damaged web diagonals No. 8 were 11 m.tons (24.3 kips), and 1,390 $kg/cm^2$ (19.7 $k/in^2$) respectively; it was therefore suggested that the increase in load on truss (from 80.7 to 114 kips) was not necessarily responsible for the extent of damage sustained by the web diagonals No. 8.

3. Roof trusses for the adjacent aisles were designed as simple trusses, but the builder made them continuous, apparently without recalculations. This kind of approach to the laws of statics resulted in the following redistribution of forces in the affected web diagonals No. 8:

| Simple truss | Continuous Truss | |
|---|---|---|
| Allowable Load | Allowable Load | Actual Load |
| Normal Force: -11 tons (-24.3 kips) | -17.2 tons (-37.9 kips) | -24.6 tons (-54.2 kips) |
| Stress: 1,390 kg/cm$^2$(19.7 k/in$^2$) | 2,170 kg/cm$^2$(30.8 k/in$^2$) | 3,100 kg/cm$^2$ (44 k/in$^2$) |

Therein lies the main cause of the damage.

Note: As a result of the study of this and one other particular case (pp.113-14 of this report) the following recommendation were made:

1. The original roof calculations should take into account whether the roof is to be exposed to industrial ash and dust deposits or not. The same set of rules should not apply in both cases.

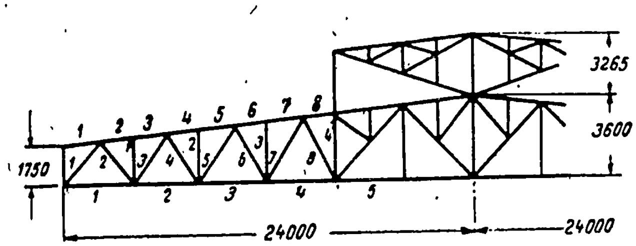
2. The previously established allowable snow loads for the Ural Region are considerably lower than the actual snow loads during the 1956/1957 winter season; in view of this, the question of roof recalculation in this region should be considered.

3. Snow should be removed from the roofs; special equipment should be designed to facilitate snow removal from extensive multi-aisle roofs.

4. Arbitrary structural design changes may prove to be dangerous; they should be avoided.

Source

Stroitel'naya Promyshlennost', No. 7, July 1957, pp. 19-21.




23.6 in. buckling occurred in web diagonal No. 8

Conversion Table

| mm. | ft. |
|---|---|
| 1,750 | 5.75 |
| 3,265 | 10.7 |
| 3,600 | 11.8 |
| 24,000 | 78.7 |

ROOF TRUSS IN AN INDUSTRIAL STRUCTURE (Middle Ural Region)
Source: Stroitel'naya Promyshlennost', No. 7, 1957, p. 20

PLATE 59

## CHAPTER XVIII

## FAILURE OF STEEL ROOF TRUSSES IN INDUSTRIAL STRUCTURES

The following study deals with a number of cases of partial or complete failure of roof trusses in industrial buildings, presumably in the Middle Ural Region.

Undertaken apparently to prevent recurrence of such failures, it deals specifically with damage due to:

a) Excessive roof dead load and snow load;
b) Excessive slenderness ratio and bends in web compression members;
c) Bends and cracks in gussets.

For convenience, the above material is treated in two parts:

1) Steel roof truss failures and their causes;

2) Gusset defects and their causes.

The examples of truss failure or damage cited in the study seem to justify this arrangement.

### I. Steel Roof Truss Failures

#### Case 1. Collapse of Roof Trusses in One-aisle Industrial Shop with Monitor.

The facts ascertained after the collapse were the following:

| | | |
|---|---|---|
| Actual weight of a 3-layer roofing | 15-35 $kg/m^2$ | ($3.07$-$7.17$ $lb/ft^2$) |
| Calculated " " " " " | 10 $kg/m^2$ | ($2.05$ $lb/ft^2$) |
| Actual thickness of roof cement-sand layer | 25-30 mm. | (0.984-1.18 in.) |
| Calculated " " " " " " | 20 mm. | (0.787 in.) |
| Actual weight of ribbed reinforced concrete panels | 105-117 $kg/m^2$ | ($21.5$-$24.0$ $lb/ft^2$) |
| Calculated " " " " | 100 $kg/m^2$( | ($20.5$ $lb/ft^2$) |
| Actual weight of truss | 50 $kg/m^2$ | ($10.2$ $lb/ft^2$) |
| Calculated " " " | 40 $kg/m^2$ | ($8.19$ $lb/ft^2$) |

Recalculations indicated that the total roof overload (dead weight and snow load) amounted to 50%, and the collapse of trusses was ascribed to that fact.

Photograph on Plate 60, fig. 1 shows the roof near monitor under a 2-2.5 m. (6.6-8.2 ft.) thick snow blanket.

Case 2. Buckling of Compressive Web Members out of the Plane of Purlin Carrier Truss.

Structure: Steel purlin carrier truss located between monitors in an industrial shop.

Construction of the Truss: The truss is shown in drawing on Plate 60, fig. 2.

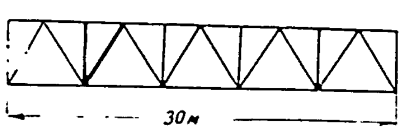
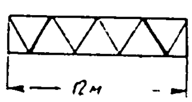
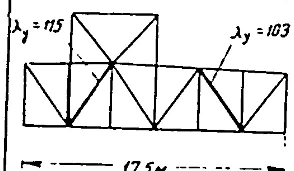
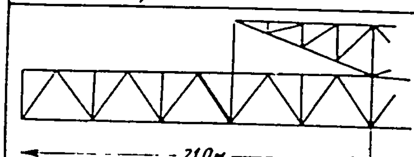
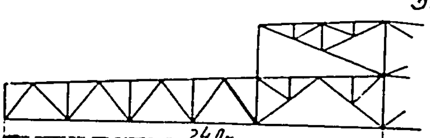
Span - 12 m (39.4 ft.)
Height - 2 m (6.56 ft.)
Upper chord - 2 angles: 100 x 75 x 8 mm. (3.94 x 2.95 x 0.315 in.)
Lower chord - 2 angles: 65 x 65 x 6 mm. (2.56 x 2.56 x 0.236 in.)
Web diagonals - a) 50 x 50 x 5 mm. (1.97 x 1.97 x 0.20 in.)
b) 65 x 65 x 6 mm. (2.56 x 2.56 x 0.236 in.)

Damage to the Truss: 2 compressive web members buckled out of the plane of the truss to the extent of 300 mm. (11.8 in.); the lower chord sagged 150 mm. (5.9 in.); the truss did not collapse because its upper chord held well in the column joint. The sagging truss is shown in drawing on Plate 60, fig. 2; part of the truss with a buckled web diagonal appears in photograph on Plate 61, fig. 1.

Causes of Damage: After the damage the following was ascertained:

| | |
|---|---|
| Slenderness ratio of buckled diagonals: | |
| a) transverse to truss plane | 80 |
| b) in the truss plane | 99 |
| Actual average snow load around truss | 360 kg/m$^2$ (73.7 lb/ft$^2$) |
| Allowable " " " " " | 140 kg/m$^2$ (28.7 lb/ft$^2$) |
| Volume weight of snow | 315 kg/m$^3$ (19.5 lb/ft3) |
| Average depth of snow blanket | 1.15 m ( 3.76 ft.) |
| Actual roof dead weight | 217 kg/m$^2$ (44 lb/ft$^2$) |
| Allowable roof dead weight | 235 kg/m$^2$( 48.2 lb/ft$^2$) |
| Actual total roof load | 580 kg/m$^2$(118.8 lb/ft$^2$) |
| Allowable total roof load | 375 kg/m$^2$( 76.8 lb/ft$^2$) |
| Overload | 36% |
| Actual force in web diagonal | 19.8 m tons (43.8 k) |
| Critical Euler force (straight diagonal) | -32.2 m tons (-71.0 k) |
| Critical Euler force (initial bends taken into account) | -22.1 m tons (-48.8 k) |
| Safety factor (minimum steel yield point - 2,400 kg/cm$^2$- 34,100 lb/in$^2$) | 22.1:19.8 = 1.12 |

The web diagonals had considerable initial bends out of the plane of the truss acquired before and/or during erection; this caused sharp decrease of critical force. Combination of this factor and snow overload produced the web buckling.

| Схемы ферм | Сечение | Длина мм | Гибкость в плоскости фермы λ | Напряжение от нормативных нагрузок σ кг/см² (с φ) при обычной методике расчета | Нормативная величина предварительного прогиба e при e/l [illegible] | Значение e при нормативных нагрузках, при котором стержень теряет несущую способность мм | Значение e/l при нормативных нагрузках, при котором стержень теряет несущую способность | Примечания |
|---|---|---|---|---|---|---|---|---|
| 1.  30м | Стойка | | | | | | | В 56 фермах было обнаружено 57 элементов решетки с e ≥ 10 мм (40% ферм (22 шт) оказались дефектными и 7,3% всех элементов решетки) Величина e доходит в элементах решетки до 40 мм |
| | ╥ 65×6 | 3400 | 111 | 1290 | 3,4 | 27,1 | 1/125 | |
| | Раскос | | | | | | | |
| | ╥ 100×8 | 4530 | 101 | 1690 | 4,5 | 16,2 | 1/280 | |
| 2.  12м | Раскос | | | | | | | Выгиб e в аварийных фермах доходит до 300 мм, в других до 150 мм, выгибы в элементах решетки равны 10 20 мм, носят массовый характер |
| | ╥ 65×6 | 2450 | 80 | 1590 | 2,5 | 10,9 | 1/225 | |
| 3.  17,5м | Стойки | | | | | | | Выгиб e до 300 мм в аварийных фермах, во многих элементах e ≥ 10 мм |
| | ┘└ 75×6 | 2900 | 100 | 1360 | 2,9 | 22,5 | 1/129 | |
| | Раскосы | | | | | | | |
| | ┐┌ 130×90×8 | 3820<br>4260 | 103<br>115 | 1440<br>1490 | 3,8<br>4,3 | 18,8<br>21,4 | 1/203<br>1/199 | |
| 4.  21,0м | Раскос | | | | | | | |
| | ╥ 60×5 | 3520 | 124 | 1330 | 3,5 | 29,1 | 1/121 | |
| 5.  24,0м | Раскос | | | | | | | Выгибы раскоса в двух аварийных фермах доходят до 0,5-0,6 м ферма работает частично как неразрезная, благодаря чему усилие в раскосе увеличивается против усилия, вычисленного по обычной методике расчета |
| | ╥ 75×6 | 3500 | 102 | 1390 | 3,5 | — | — | |

Case 3. Buckling of a Post and Two Diagonals in a Roof Truss.

In this case, the snow load was below that allowed; the total load on the truss was only 92% of that allowed.

Nevertheless, two web diagonals buckled out of the plane of the truss to the extent of 300-500 mm. (11.8-19.7 in.) and so did one of the posts.

The damage was caused by initial bends out of the plane in those members, which were inflicted upon them in the course of transporation and erection.

The drawing of the truss with some pertinent data appear in the table below (truss No. 3); the extent of damage may be seen in Photograph on Plate 61, fig. 2. Some data pertaining to damaged steel trusses, already discussed or similar to them (except trusses No. 1 and No. 4), are presented in the table that follows:

Data on Damaged Steel Roof Truss.

| Truss Span (L) Feet | Cross section of the member Inches | Length of the member. Feet | Slenderness ratio transverse to the plane of truss. | Standard compressive stress, including direct stress $P/A$ & bending stress $Pe/S$ ($\phi$ x straight-member stress). lb/in.$^2$ | Fabrication tolerance ($e/l$=1/1000) in. |
|---|---|---|---|---|---|
| 98.4 | | POST | | | |
| | 2.56x2.56x0.236 | 11.2 | 111 | 18,300 | 0.134 |
| | | DIAGONAL | | | |
| | 3.94x3.94x0.315 | 14.9 | 101 | 24,000 | 0.177 |
| | | DIAGONAL | | | |
| 39.4 | 2.56x2.56x0.236 | 8.04 | 80 | 22,600 | 0.098 |
| | | POST | | | |
| 57.4 | 2.95x2.95x0.236 | 9.5 | 100 | 19,300 | 0.114 |
| | | DIAGONALS | | | |
| | 5.1x3.54x0.315 | 12.5 | 103 | 20,400 | 0.150 |
| | | 14.0 | 115 | 21,200 | 0.169 |
| | | DIAGONAL | | | |
| 68.9 | 2.36x2.36x0.197 | 11.5 | 124 | 18,900 | 0.138 |
| | | DIAGONAL | | | |
| 78.7 | 2.95x2.95x0.236 | 11.5 | 102 | 19,700 | 0.138 |

a) The damaged members are indicated by heavier lines on truss drawings;
b) $e$ -value in inches of deflection ordinate in a compressed member bent under axial load;
c) $\phi$ -coefficient of decrease in axial compression allowable stress when both the direct compression and bending stress are taken into account.

| Critical value of e at which combined stresses exceed allowable | Eccentricity ratio $e/l$ for complete loss of strength | R E M A R K S |
|---|---|---|
| 1.07<br>0.638 | 1/125<br>1/280 | 57 web members with $e \geq$ 0.394 in. in 56 trusses (in some cases =1.58 in.)<br>Some 40% of trusses (22 out of 56) and 7.3% of all web members defective. |
| 0.429 | 1/225 | Bend $e$ in failed trusses $\leq$ 11.8, in other trusses $\leq$ 5.9 in.; $e$ in web members = 0.394 -0.787; widespread |
| 0.886<br>0.740<br>0.843 | 1/129<br>1/203<br>1/199 | $e$ in failed truss $\leq$ 11.8 in.; in numerous web members $e \geq$ 0.394 in. |
| 1.15 | 1/121 | |
| ---- | | $e$ in web diagonals in 2 failed trusses $\leq$ 19.7-23.6 in. Trusses function as partly continuous; the force in damaged diagonals is therefore greater than calculated. |

It is evident from the table that the allowable values of $e(e/l - 1/1000)$ of 2.5 - 4.5 mm. (0.1-0.17 in.) were exceeded and the stresses from allowable loads were close to 1,600 kg/cm$^2$ (22,700 lb/in$^2$).

## II Gussets: Defects and Their Causes

Cracks in or breakage of gussets are apparently due to the quality of steel they are made of and to careless handling of the trusses.

Examination of roof structure in four shops of an unspecified plant established the facts summarized in the table below:

**Results of Examination of Trusses and Gussets in Four Shops of One Plant.**

| Shop | No of trusses | No of cracks in gussets | No of trusses with cracks | % of trusses with cracks | % of joints with cracks | No of cracks in craters | No of trusses with cracks in craters | % of trusses with cracks in craters | % of welded joints with cracks | Remarks |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 126 | 30 | 22 | 17.5 | 1.2 | 228 | 74 | 59 | 0.90 | |
| 2 | 260 | 13 | 13 | 5 | 0.3 | 164 | 78 | 30 | 0.35 | |
| 3 | 62 | 16 | 15 | 24 | 1.4 | 112 | 51 | 82 | 1 | |
| HALF TRUSSES | | | | | | | | | | |
| 4 | 70 | 22 | 14 | 20 | 3 | 206 | 35 | 50 | 3 | Cracks extended to metal |

The table indicates that the number of trusses with cracks in this particular plant amounts to 5-24%, and the number of gussets with hot cracks in welded joints to 0.3-3%.

As a rule, the trusses (web members and gussets) are fabricated of Mark St. 3 Martens rimming steel (ultimate strength - 54,000 lb/in$^2$; yield point - 34,000 lb/in$^2$).

In the case under consideration, chemical and mechanical tests of specimens indicated that the material satisfied the GOST requirements. Metallographic examination, however, disclosed that steel contained high percentage of impurities. Moreover, impact strength tests gave the following low and non-uniform, scattered results:

at - 10°C (14°F), the impact strength amounted to: 0.4; 0.6; 4.2; 6.5 kgm/cm$^2$
(18.7; 28.0; 196.0; 303.0 ft-lb/in$^2$)

at + 20°C (68 F), after mechanical aging, the impact strength amounted to 0.5 - 2.3 kgm/cm$^2$ (23.4 - 107 ft-lb/in$^2$)

Under the impact of dynamic, repeated static and vibrational loads, the presence of slag impurities as well as of nitrogen, hydrogen and oxygen in Mark St. 3 rimming steel sharply lowers the ductility of steel at low temperatures and increases its tendency to form cracks, particularly at the ends of the joints.

In numerous cases, gussets broke or cracked because of the bends produced in them during erection, particularly in winter time. As a rule, the cracks were parallel to the chord and started at the end of the welds joining the web diagonals (Plate 60, fig 3); they appeared after the bends were made in the gussets during the lifting or dragging of trusses lying flat on the ground.

It is recorded that when, at a temperature of - 12°C (10.4°F), a crane carrier truss was once lifted by its upper chord, all three gussets at the lower chord broke, and the lower chord broke away from the truss. Also recorded were cases of breakage of gussets to which were fastened web members of half-trusses with the far ends free. This breakage was due to the repeated thrusts and/or blows on the edge of those members in the course of the loading of the half-trusses and their transportation, unloading, moving around the building site and field assembly.

## Recommendations

Study of failures and defects, cited above, resulted in the following recommendations:

1. Dead weight and snow loads: The following coefficients are currently (1957) adopted in accordance with the norms for structural design:

| | |
|---|---|
| Dead weight overload | 1.1 |
| Dead weight overload for heat insulation slabs and filling | 1.2 |
| Adjustment coefficient to allow for working conditions of roof truss compressive members (NiTU) | 0.95 |
| Snow overload | (unspecified) |

Considering that truss web members are as a rule made of comparatively small angles (sections of a number of truss members are selected without safety margin) and may be easily bent or indented, the adoption of the following coefficients was recommended:

| | |
|---|---|
| Roof dead load overload | 1.2 |
| Adjustment coefficient to allow for working conditions for compressive web members | 0.8 |
| Snow overload coefficient increased at least to | 1.6 |

The specific gravity of snow may be considerably higher than that adopted; moreover, one should not lose sight of the fact that on many shops with extensive roof areas thousands of tons of snow may sometimes accumulate in the course of a few hours.

2. Maximum slenderness ratio for two-angle compressive web members. The higher the slenderness ratio, the larger will be the deflection ordinate under the load. Considering that in the last few years the values of allowable stresses were increased and that web members are made of small angles, the value of the maximum slenderness ratio out of the plane of truss should be limited to 70-80 for the compressive members of the truss.

3. Maximum value of the initial deflection ordinate transverse to the plane of the truss for web struts. Frequently, the presently established ratio of =1/1000 cannot be maintained because of the bending sustained by the truss web members (assuming they were properly fabricated in the first place) in the course of transportation of trusses and their erection. It is therefore suggested that:

a) web struts be made of stamped or bent structural steel (without increasing the weight of trusses) instead of small steel angles;

b) adjustment coefficient to allow for working conditions of web struts be lowered to 0.8.

4. Increased toughness and stiffness of gussets. In the process of manufacturing, transportation, and erection of trusses, the gussets may be subjected to dynamic loads and plastic deformation from repeated static loads. In view of that, the steel plate from which gussets are made should have the following impact strength:

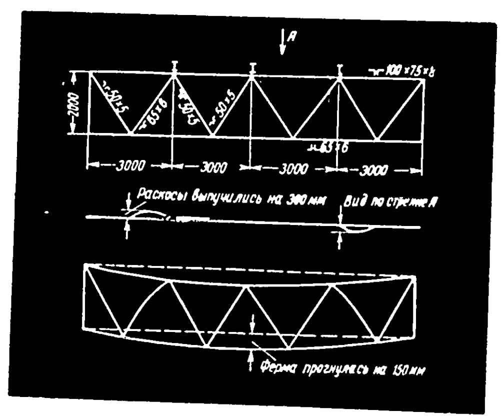
at + 20°C (68°F) - not lower than 7 kgm/cm$^2$(327 ft-lb/in$^2$)
at + 20°C (68°F) - during aging-not lower than 3-4 kgm/cm$^2$ (140-186.5 ft-lb/in$^2$)
at - 30°C (22°F below 0) not lower than 3-4 kgm/cm$^2$ (140-186.5 ft-lb/in$^2$)

Source

Stroitel'naya Promyshlennost' No. 5, 1957, pp. 22-27.

Fig. 1. 6-8 ft. snow blanket near monitor (roof trusses collapsed)

Fig. 2. Purlin carrier truss (buckling of compressive web members). Two diagonals buckled 11.8 in. transversely to the plane of the truss. Chords sagged 5.9 in.




Conversion Table

| mm. | ft. |
|---|---|
| 2,000 | 6.56 |
| 3,000 | 9.84 |

Fig. 3. Cracked gusset

Angles

| mm. | in. |
|---|---|
| 50 x 50 x 5 | 1.97 x 1.97 x 0.197 |
| 65 x 65 x 6 | 2.56 x 2.56 x 0.236 |
| 100 x 75 x 8 | 3.94 x 2.95 x 0.315 |

STEEL ROOF TRUSSES IN INDUSTRIAL STRUCTURES (Middle Ural Region)
Source: Stroitel'naya Promyshlennost', No. 5, 1957, pp. 22,23,25.

PLATE 60

Fig.1. Purlin carrier truss (1-Buckled web diagonal)

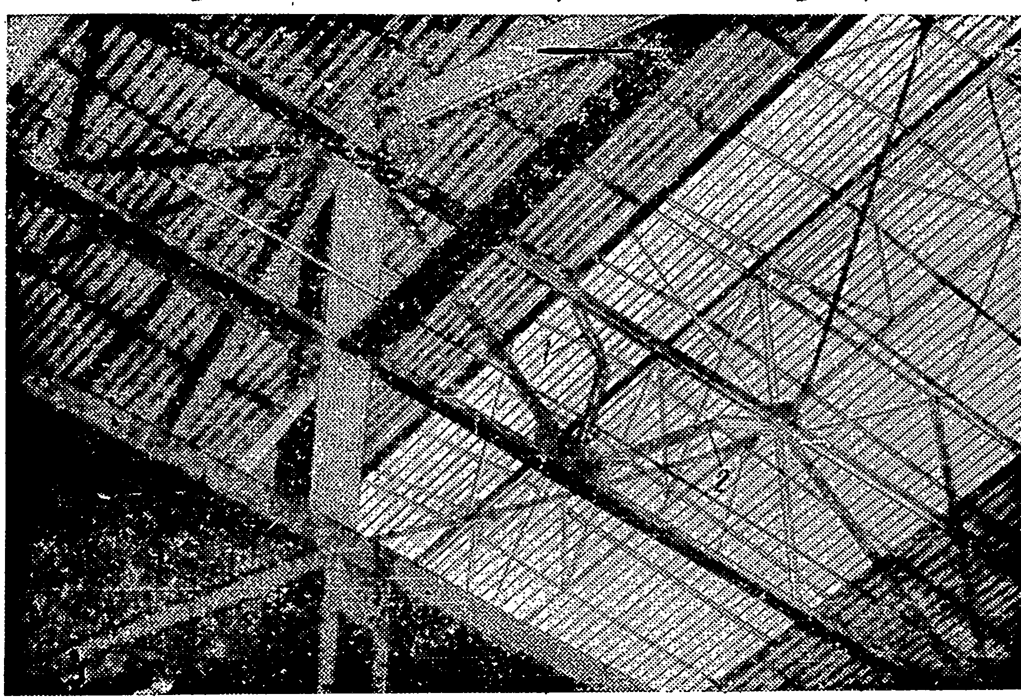


Fig. 2. Roof truss (1-Buckled post; 2-Buckled web diagonal)

STEEL ROOF TRUSSES IN INDUSTRIAL STRUCTURES (Middle Ural Region)
Source: Stroitel'naya Promyshlennost', No. 5, 1957, pp. 23, 24

PLATE 61



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# CONCLUSION

The few cases of structural failures and defects analyzed in the preceding chapters provide some insight into the workings of the mind of the builders carrying out the construction programs of the Soviet Union; they also indicate that:

1. There are certain inaccuracies and gaps in the Soviet building codes;
2. The designers make mistakes at times;
3. Defective or wrong materials are sometimes used in construction;
4. There appear to be unsolved problems in connection with freezing soil and winter construction;
5. The builders are frequently negligent; they are just as apt to disregard the building code rules as to defy the dictates of common sense.

All this cannot but affect the soundness as well as the appearance of Soviet structures and seems to be quite in line with the impressions gained by casual foreign travelers in the Soviet Union. Of particular interest in this respect, should be the impressions of one such traveler, Albert Gore, U. S. Senator from Tennessee. During his visit to the Soviet Union in 1957, he was taken on an inspection tour of the 5,000 kw. experimental reactor (about 65 miles S. W. of Moscow) and the Cancer Research Institute. Here is what he says*:

> "...Although only 5 years old, the building looked many years older. The woodwork was awry, the plaster cracked, paint daubed and bathroom facilities antiquated. The reactor engineering, however, was of an entirely different sort ....Next we visited the Cancer Research Institute. Here too, the buildings were poorly designed** and constructed, with even poorer workmanship. The instruments for isotopic treatment, however, appeared to be of excellent quality..."

Significant as such reports by foreign visitors may be, the following question still remains: how true is the qualitative picture of Soviet construction as a whole, drawn on the basis of a few such reports plus some twenty odd cases of failures analyzed in this report? There is a temptation to jump at uncomplimentary conclusions which could prove to be too one-sided. Under these circumstances, it would be more judicious to turn to some Soviet functionary directly connected with the building industry for an opinion, untinged by propaganda, and candid, if possible.

*"Senator Gore Reports". Herald Tribune, Sunday, 20 October 1957.
**Could Senator Gore have meant "designed" in the sense of "laid out"?

Strangely enough, such a functionary seems to exist, and here is what he has to say*:

"A certain improvement in the quality of construction work in cities and workers' villages has been achieved in recent years. Numerous building organizations have begun to build faster because of the accumulated experience as regards sectional construction and the mechanization of construction work. In a number of cities (Leningrad, Rostov on Don, etc.) good houses are built and much has been done with respect to public services in residential sections.

In spite of certain progress, the quality of construction work as a whole remains unsatisfactory. Buildings with many defects and unfinished details are offered to the State Commissions for acceptance. In many cities, the houses not accepted by the State Commissions are invaded by tenants, without authorization; but on the other hand, such invasions take place in numerous cases with the blessings of the City Executive Committees. Flagrant violations of technical conditions have occurred in the structural work procedures; for this reason, for instance, two failures took place in 1956 at the building sites of the "Ufastroy" Trust of the Ministry of Urban and Rural Construction of the RSFSR. The above weaknesses in construction procedures were also observed in a number of other cities.

The basic causes underlying the low quality of construction are the following:

1. Inadequate qualifications of the workers and their extensive turnover;
2. Low quality of building materials and "products" (possibly hardware, sub-assemblies, etc....);
3. Lack of good quality tools;
4. Violation of "Technical Conditions" in construction work;
5. Lack of proper control on the part of City Executive Committees and local "Construction and Architecture" agencies;
6. Failure to exact adequate standards on the part of local authorities, the "Gosarkhstroykontrol" (State Architectural and Construction Control) and the technical supervisors of the client;
7. Incorrect planning of construction;
8. Lackadaisical handling of construction work during the first three quarters of the year and a fitful rush in the last quarter (for example, in 1955, at Ufa, it was in the 4th quarter that 76% of the construction called for by the yearly program was turned over to the operating agencies; at Sverdlovsk, it was 63%, but 36% of that was in December)".

So writes a Soviet functionary. In the main, the above statement apparently concerns residential structures, but the data in this report indicate that it applies, to some degree at least, to industrial structures as well.

*"Construction Quality Control". Byulleten' Stroitel'noy Tekhniki (Bulletin of Building Engineering), No. 2, 1957, p. 43.

At this point, it may be well to recall that Senator Gore while criticizing Soviet buildings had some words of praise for the equipment they contained. This equipment belongs in the domain of barely explored branches of modern science. If the Soviet engineers are capable of progress in the field of modern science, it may be assumed that Soviet builders, potentially at least, are competent properly to apply the principles of such an ancient science as the science of building. But this, it appears, is a problem to be solved in an indefinite future. For the present, one is inclined to conclude that the Soviet building industry presents a sad picture. The Soviet functionary has implied that much.

Under the Soviet scheme of things the question of priorities appears to be of paramount importance. It does not matter that workers live in crumbling residential structures and work in unsoundly built industrial structures. What seems to matter is the perfection of some apparatus they manufacture under a priority even if, upon its completion, this apparatus is installed in a structure of poor workmanship.

The Soviet government thinks in terms of grandiose economic expansion at home and competition with the United States abroad; moreover, it is confronted with submissive but nonetheless housing-hungry population (there is evidence of this even in this report - unauthorized occupancy of unaccepted buildings). In these circumstances, the government may cease issuing orders to the building industry, orders which it does not seem strictly to enforce. Instead, the industry may be put on a priority list, and its uneconomic performance may be thus brought to an end.